CHAPTER 5.6

CONCRETE BENT CAPS

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5.6.1 INTRODUCTION

A concrete bent consisting of columns and a bent cap beam is an intermediate support between bridge spans that transfers and resists vertical loads and lateral loads such as earthquake and wind from the superstructure to the foundation. The bent cap beam supports the longitudinal girders and transfers the loads to the bent columns.

A typical elevation view of a concrete bent integrally connected with the superstructure is shown in Figure 5.6.1-1.

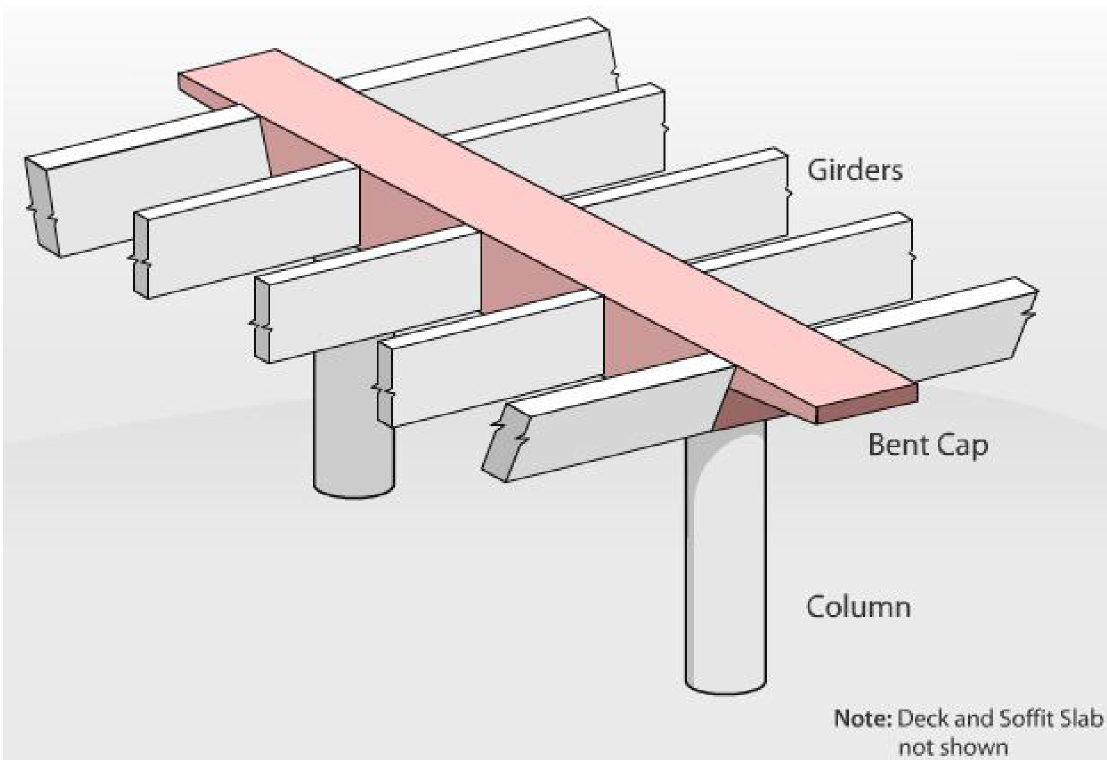


Figure 5.6.1-1 A Typical Integral Concrete Bent

Bents can be classified as a single-column, a two-column, or a multicolumn bent as shown in Figure 5.6.1-2.

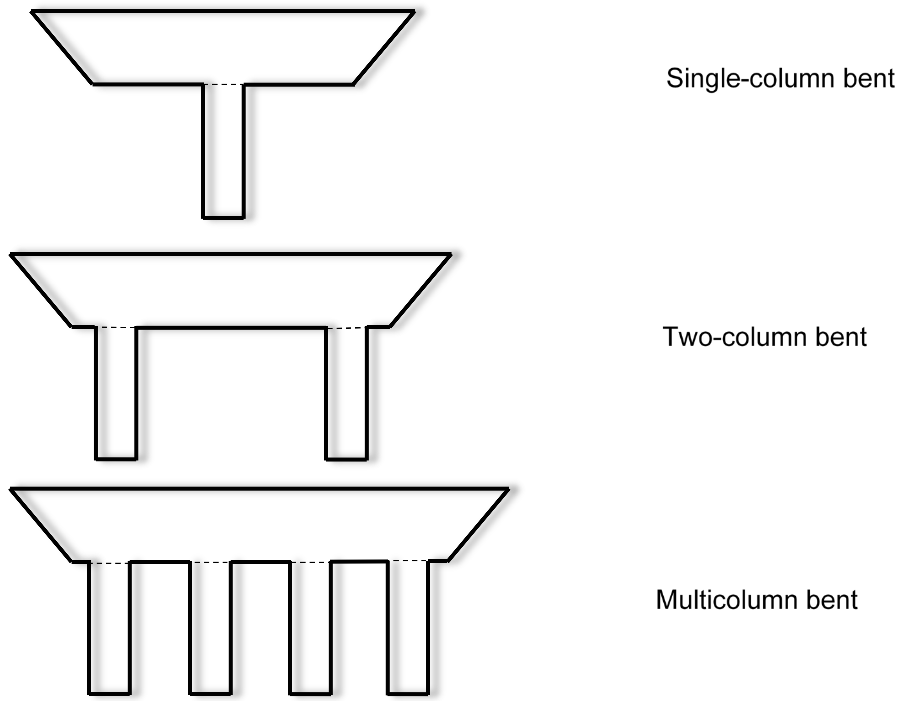


Figure 5.6.1-2 Typical Bents

In this chapter typical concrete bent caps are discussed. Design considerations for the drop bent cap, the integral bent cap, and inverted tee bent cap are presented. Two design examples including a reinforced concrete integral bent cap and a drop bent cap are given to illustrate the design procedure.

5.6.1.1 Types of Bent Caps

The typical types of bent caps are:

- Drop bent cap
- Integral bent cap
- Inverted tee bent cap

These bent caps may be configured in conventional bent types as shown in Figure 5.6.1-2 or may possess asymmetric column configurations. Unusual bent types include "C" bents, and outrigger bents.

5.6.1.1.1 Drop Bent Cap

A drop bent cap, as shown in Figure 5.6.1-3, supports the superstructure girders directly on its top. This type of the bent cap is generally used when the superstructure consists of precast concrete or steel girders.

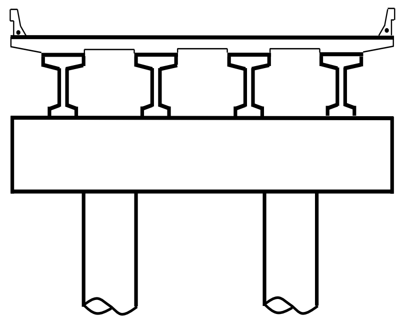


Figure 5.6.1-3 Overview of Drop Bent Cap

Drop bent caps may have different types of connections to the superstructure diaphragm: fixed or pinned. Drop caps with pinned (shear key pipe or high strength rod) connections connection are most often used.

5.6.1.1.2 Integral Bent Cap

An integral bent cap, as shown in Figure 5.6.1-4, is cast monolithically with the superstructure girders, and typically has the same depth as the superstructure. The superstructure girders are framed into the bent cap and are supported indirectly by the bent cap. This type of the bent cap is commonly used in cast-in-place concrete box girder construction. It also can be used for a steel girder bridge to provide a longitudinal frame action, as shown in Figure 6.2.1-1 in Bridge Design Practice (BDP) Chapter 6.2. The load from the girders is transferred as point loads along the length of the bent cap.

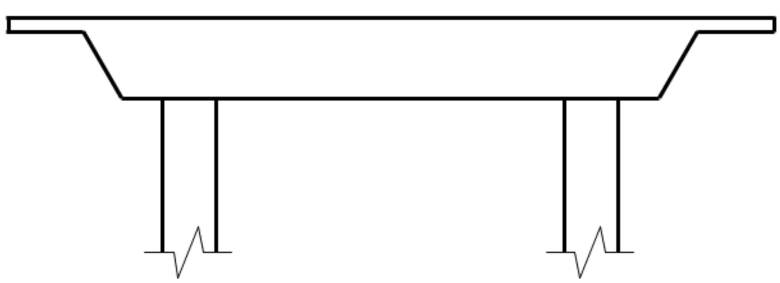


Figure 5.6.1-4 Integral Bent Cap

As a monolithic connecting element between columns and girders, reinforcement details in integral bent caps can be challenging. Figures 5.6.1-5 to 5.6.1-8 show three-dimensional schematics of bar reinforcement from the superstructure that must be accommodated by the integral bent cap. The integration of bar reinforcement from the columns should be considered carefully.

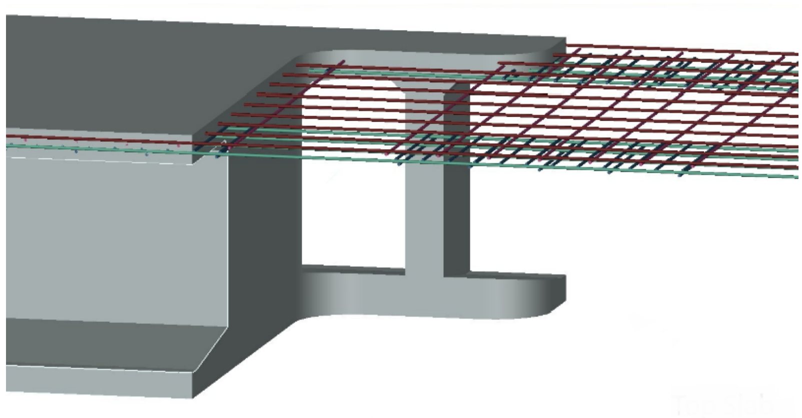


Figure 5.6.1-5 Integral Bent Cap Top Slab Reinforcement

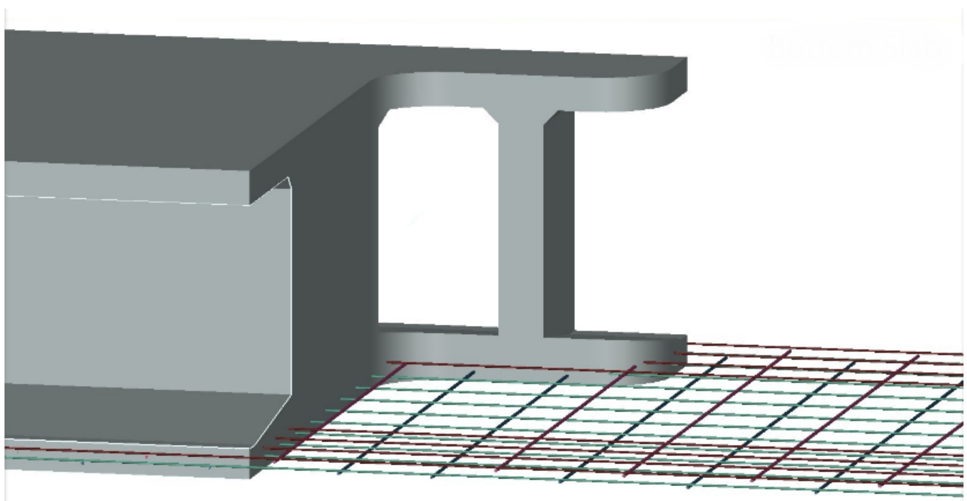


Figure 5.6.1-6 Integral Bent Cap Bottom Slab Reinforcement

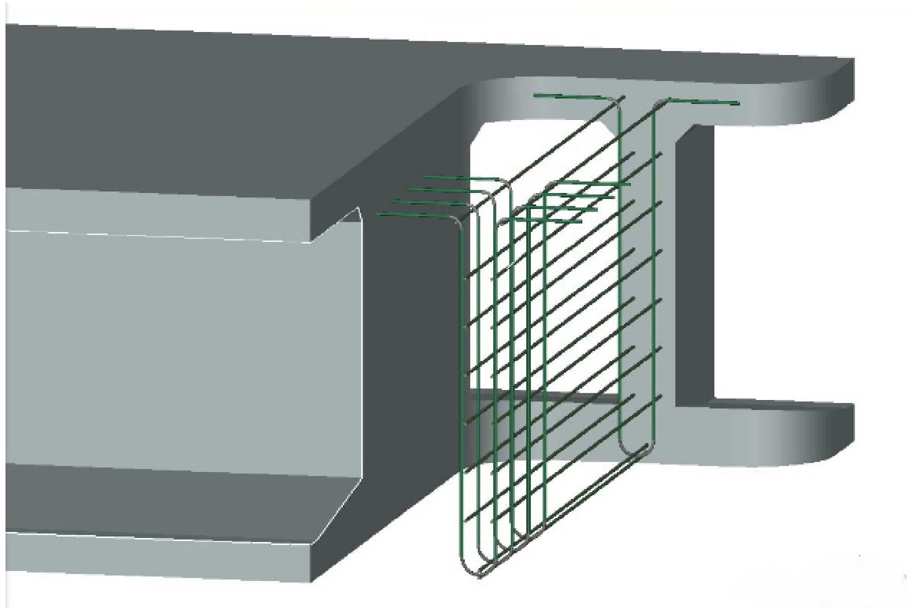


Figure 5.6.1-7 Integral Bent Cap Girder Reinforcement

Note: For clarity, post-tensioning ducts are not shown.

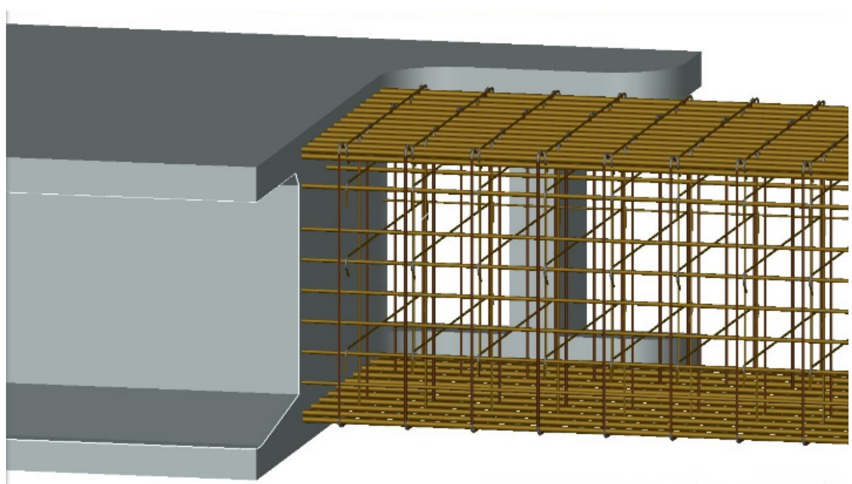


Figure 5.6.1-8 Bent Cap Reinforcement of Integral Bent Cap

5.6.1.1.3 Inverted Tee Bent Cap

The inverted tee cap, as shown in Figure 5.6.1-9, is commonly used with precast concrete girders to accommodate vertical clearance within the traveled way and to enhance constructability and aesthetic appearance. However, from a design standpoint, it is difficult to satisfy seismic demands. The reinforcement of the seat of the inverted tee cap also presents special challenges in shear, flexure, and bar anchorage.

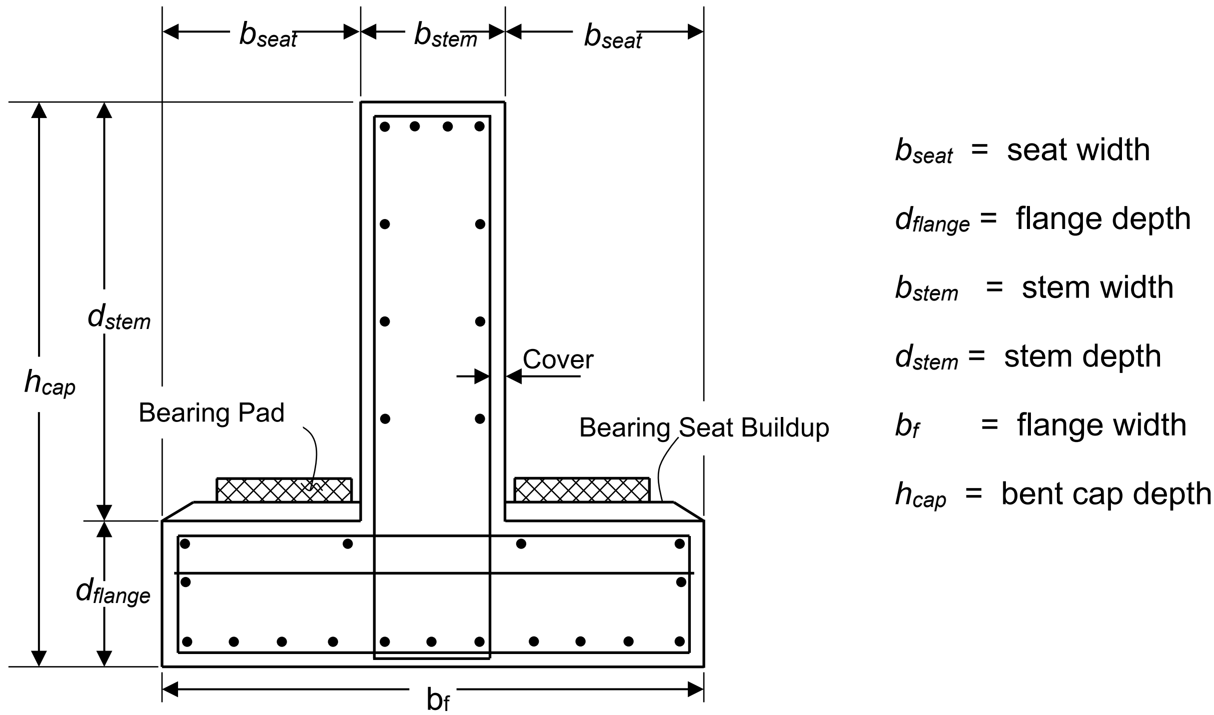


Figure 5.6.1-9 Inverted Tee Bent Cap

5.6.1.2 Member Proportioning

The bent cap depth should be deep enough to develop the column longitudinal reinforcement without hooks in accordance with Caltrans Seismic Design Criteria (SDC) 7.3.3 and 8.3.1 (Caltrans, 2019b). For integral bent caps, the minimum bent cap width required by SDC 7.4.3 for adequate joint shear transfer shall be the column sectional width in the direction of consideration, plus two feet.

Drop caps that support superstructures with expansion joints shall have sufficient width to prevent unseating. In accordance with SDC 7.3.2.2, the minimum width for non-integrated bent caps is determined by considering displacement of the superstructure due to prestress shortening, creep and shrinkage, thermal expansion and contraction, and earthquake displacement demand.

For inverted tee bent caps, the stem should be a minimum three inches wider than the column to allow for the extension of column reinforcement into the cap. Similar to the integral and drop caps, the depth of the inverted tee bent cap should be adequate to develop the column bar reinforcement without hooks. The seat width (b_{seat}) should be adequate for the primary flexural reinforcement to develop fully.

A bent cap shall be designed as either the sectional design model (conventional beam) or the Strut-and-Tie (deep beam) in accordance with Article 5.7.1.1. The designer may use the legacy methods if all the criteria listed on AASHTO-CA BDS-8 Articles 5.8.4.1 are met. Refer to BDP Chapter 5.1 for details.

5.6.2 LOADS ON BENT CAPS

This section discusses the type of loads applied on bent caps.

5.6.2.1 Permanent Loads

Permanent loads and forces are either constant or varying over a long period of time after construction is completed.

For bent cap design, the permanent loads include:

- Dead load of structural components and nonstructural attachments (*DC*):

    1. Bridge weight: In cast-in-place box girder superstructures, normal weight concrete is 150 pcf, including the weight of bar reinforcing steel and lost formwork. In precast concrete girder or steel girder superstructure using cast-in-place concrete decks, weight of concrete decks between precast concrete and steel girder flange edges shall be increased by 10 percent to account for the weight of the stay-in-place metal form and additional concrete in the form corrugations in accordance with Article 3.5.1 CA Amendment.
    2. Weight of barrier
    3. Weight of any other permanent attachments, such as sound walls or sign structures

- Dead load of wearing surfaces and utilities (*DW*), including all dead loads added to the bridge after it is constructed:

    1. For new bridges, 35 pounds per square foot shall be applied on the bridge deck between the faces of barrier rails to account for three inches of future wearing surface in accordance with Article 3.5.1 CA Amendment.
    2. Weight from utilities: For example, a 24-inch water line would consist of a uniformly distributed load from the pipe, hardware, and support blocks, as well as the water conveyed in the line.

- Force effects due to creep (*CR*): *CR* are a time-dependent phenomenon of concrete structures due to sustained compression load. As such, bent caps are generally not affected by the displacement-generated loads unless they are prestressed.
- Force effects due to shrinkage (*SH*): The *SH* of concrete structures are a time-dependent phenomenon that occurs as the concrete cures. The effects of shrinkage are typically not considered unless the bent cap is unusually long (wide structures). Shrinkage, like creep, also affects prestressed bent caps by creating a loss in prestress force as the structural member shortens beyond the initial elastic shortening.

- Secondary forces from post-tensioning (*PS*): The primary post-tensioning forces counteract dead and live load demands. However, *PS* forces introduce load into the members of statically indeterminate bent caps as the cap beams shorten elastically toward the point of no movement.
- Miscellaneous locked-in force effects resulting from construction processes (*EL*)

Generally, *DC* and *DW* are distributed by tributary area (or width) for precast prestressed I-girder, CA wide flange, bulb T girder, and steel girder bridges. In other types of structures, *DC* and *DW* may be distributed equally to each girder despite varying girder spacing. Those types of structures, such as cast-in-place prestressed concrete box girder sections, are so stiff that dead loads are distributed nearly equally to each girder. The self-weight of the bent cap, $DC_{bent\ cap}$, however, is distributed along the length of the bent cap as a distributed load.

5.6.2.2 Transient Loads

Transient Loads are loads and forces that are, or are assumed to be, varying over a short time interval. A transient load is any load that will not remain on the bridge indefinitely. For bend cap design, this includes vehicular live loads *(LL*) and their secondary effects including dynamic load allowance (*IM*), braking force (*BR*), and centrifugal force (*CE*). Additionally, there may be pedestrian live load (*PL*), force effects due to uniform temperature (*TU*), and temperature gradient (*TG*), force effects due to settlement (*SE*), water load and stream pressure (*WA*), wind load on structure (*WS*), wind on live load (*WL*), friction load (*FR*), ice load (*IC*), vehicular collision force (*CT*), vessel collision force (*CV*), and earthquake load (*EQ*).

The primary transient load for the bent cap is vehicular loads. Force effects from live loads are determined similarly to the methods used for the longitudinal girder analysis—through the use of an analytical process that may involve influence lines. The process of calculating wheel line loads to apply to the bent cap model involves extraction of the unfactored bent reactions for each design vehicle class from the longitudinal analysis model. Note that the reactions are generated for a single truck or lane load for each of the three vehicle classes: HL-93, Caltrans permit vehicles (LL_{permit}) (Caltrans, 2019a) herein (P-15), and fatigue vehicle.

5.6.2.2.1 Number of Live Load Lanes

Live load lanes (Figure 5.6.2-1) are not the same as the striped lanes on bridges. For bent design, force effects from a single lane of the vehicular live load are acquired from the longitudinal frame analysis. To perform an analysis at the bent, various configurations of a single lane or multiple lanes are considered. Fractional lanes are not allowed for bent cap design, meaning only whole numbers of 12-ft lanes are employed.

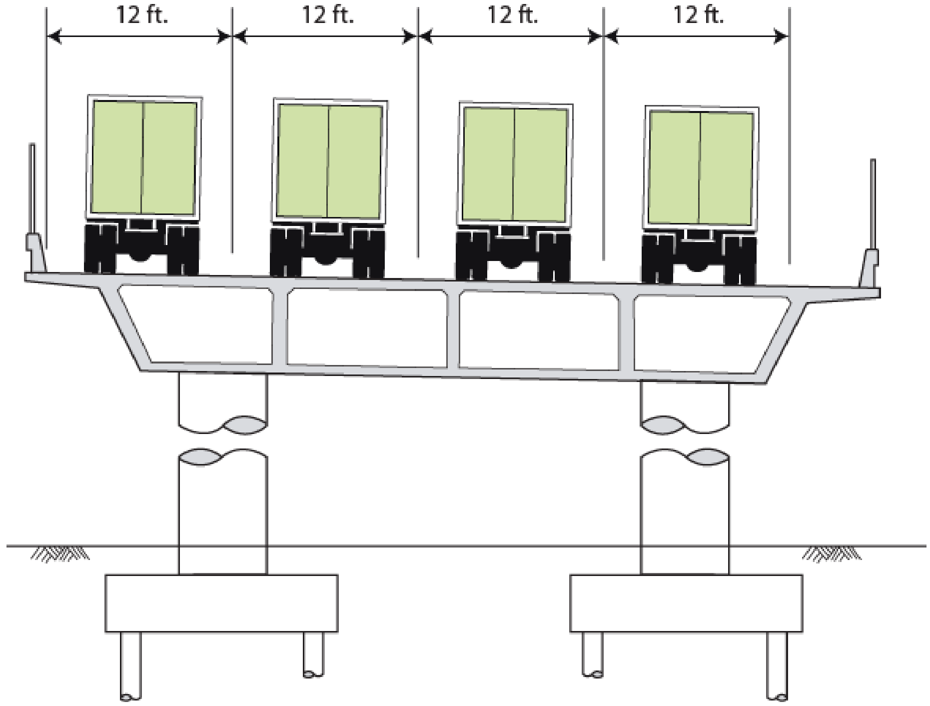


Figure 5.6.2-1 Number of Live Load Lanes

Maximum number of live load lanes within a bridge, N, is determined by following equation:

$$N = \text{Integer part of } \{[\text{width of bridge (ft)} - \text{barrier widths (ft)}] / 12 \text{ ft}\} \qquad (5.6.2\text{-}1)$$

Per Article 3.6.1.1.1, future changes to clear roadway width should be considered. The lane load is considered uniformly distributed over a 10-ft width. However, designers may simplify the analysis by combining the HL-93 lane load with the HL-93 truck wheel line load. Note that, per Article 3.6.1.2.4, the dynamic load allowance, *IM*, shall be applied only to the truck load.

5.6.2.2.2 Multiple Presence Factors, *m*

Multiple presence factors, *m*, as specified in AASHTO Table 3.6.1.1.2-1, are used to account for the improbability of fully loaded trucks crossing the structure simultaneously and are applied to the vehicular live loading.

5.6.2.2.3 Vehicular Live Load Positioning

An important consideration in the design of bent caps is to determine the maximum or critical force effects by positioning live load lanes. The different locations of the truck and lane, as shown in Figure 5.6.2-1, has different force effects on the bent cap. Whether the truck is at midspan of a bent cap or at the support, it has an effect on the value of the moment and shear on the bent cap.

AASHTO-CA BDS-8 requires that one vehicle per lane be placed in the bridge at a time. If the bridge can fit four lanes, then up to four trucks can be placed on the bridge, one in each lane.

Lanes should be placed to produce the maximum force effects in the bent cap. However, one shall consider the effect of the multiple presence factor, *m*, as it is not always evident that placing the maximum number of trucks will produce the maximum force effects in the bent cap. For example, for closely spaced columns within a bent cap, two HL-93 lanes may result in greater shear effects than four HL-93 lanes because the latter case has would require a multiple presence factor of 0.65.

It should be noted that a certain configuration, and number of live load lane positions may result in maximum shear effects but not necessarily maximum moment effects. For a bent cap supported by multiple columns, it is advisable to use a structural analysis program that is capable of generating combinations of lane configurations, as well as influence lines from moving live loads. CSiBridge and CTBridge are such programs.

5.6.2.2.4 HL-93 Design Vehicular Live Load Positioned Transversely

The HL-93 consists of a combination of the design truck, or design tandem, and design lane load. Figure 5.6.2-2 shows one of two alternatives for a design truck, or wheel lines, transversely placed within a 12-ft live load lane. The other alternative is a mirror image of this graphic depiction. The wheel lines may move anywhere within the 12-ft lane as long as Article 3.6.1.3.1 is satisfied. Lanes and wheel lines shall be placed to produce maximum force effects in the bent cap.

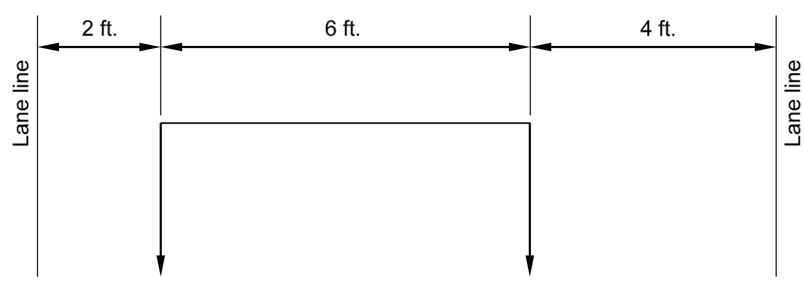


Figure 5.6.2-2 HL-93 Design Truck Positioned Transversely

When multiple lanes are applied to the bent cap, the wheel lines may be positioned as shown in Figure 5.6.2-3:

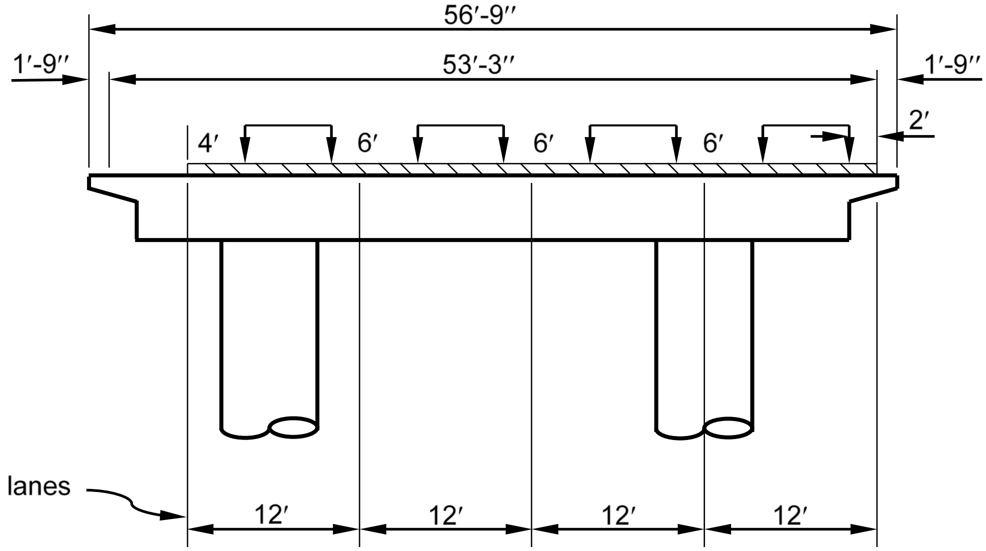


Figure 5.6.2-3 Wheel Line Spacing for Four HL-93 Trucks

Figure 5.6.2-4 shows a 10-ft wide HL-93 lane load placed in 12-ft wide lanes.

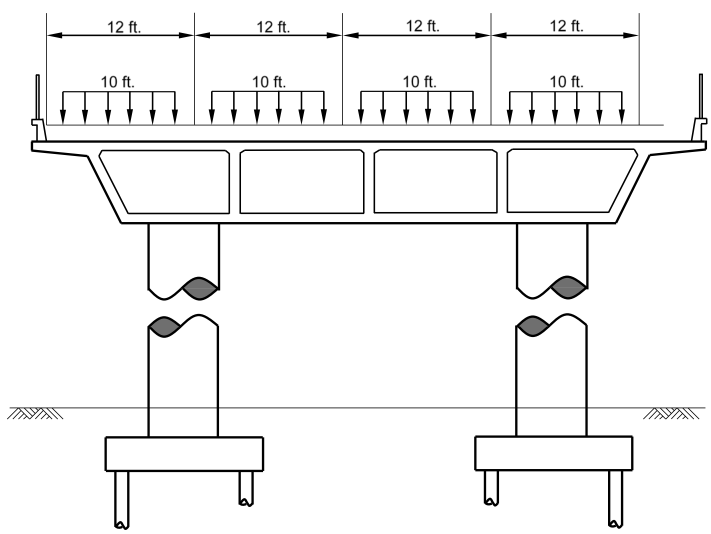


Figure 5.6.2-4 HL-93 Lane Loads Positioned Transversely

5.6.2.2.4 Permit Trucks Positioned Transversely

Per CA 3.6.1.8.2 (Caltrans 2019a), for bent cap design, a maximum of two permit trucks shall be placed in lanes that are positioned to create the most severe condition. Figure 5.6.2-5 shows two permit trucks occupying two adjacent lanes. However, the lanes may be positioned apart if that results in maximum bent cap force effects.

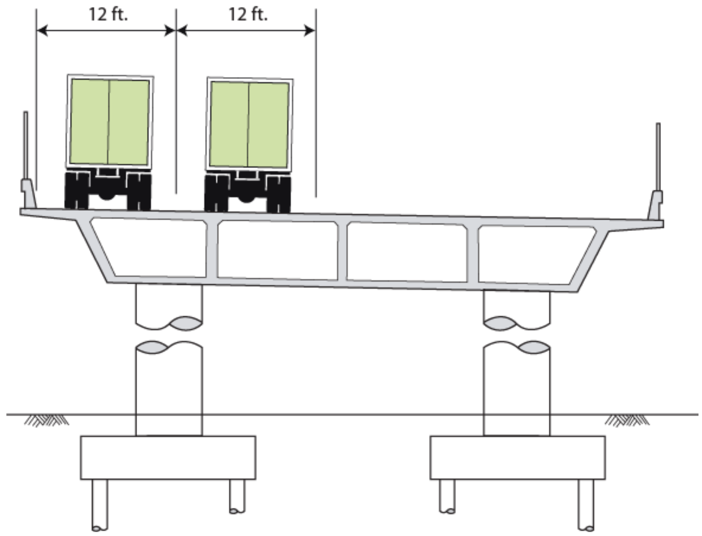


Figure 5.6.2-5 Permit Trucks Positioned Transversely

5.6.3 DESIGN CONSIDERATIONS

This section presents the typical design procedure for cast-in-place concrete bent caps. Topics include design for flexure, shear, and shear-flexure interaction.

The concrete bent cap shall be designed in accordance with AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019a) and SDC (Caltrans, 2019b).

5.6.3.1 Flexural Design

Concrete bent caps shall be designed to satisfy the strength, service, and fatigue limit states.

The goal of flexural design under the strength limit state is to provide enough resistances to satisfy the strength limit state conditions. This may be achieved by using bar reinforcing steel or prestressing in the cast-in-place concrete bent cap.

5.6.3.1.1 Flexural Design Process

The flexural design process for bent caps consists of 13 primary steps, summarized below (some steps may not apply for the reinforced concrete bent cap):

1. Calculate factored moments for strength limit states
2. Calculate minimum design moment, M_{min}
3. Determine the factored moment, M_u
4. Assume an initial value for area of nonprestressed tension reinforcement, A_s
5. Assume resistance factor, ϕ

6. Determine whether the section is rectangular or flanged
7. Calculate the average stress in prestressing steel, f_{ps}, if the bent cap is post-tensioned
8. Calculate the nominal flexural resistance, M_n
9. Calculate the factored flexural resistance, M_r
10. Calculate net tensile strain, ε_t, and check resistance factor, ϕ, assumption
11. Iterate steps 4 through 9 until $M_r \geq M_u$ and the design assumptions are verified
12. Check for serviceability
13. Check for fatigue

- *Calculate Factored Moments for Strength Limit States*

  Per CA table 3.4.1-1, Strength limit states are used to calculate M_u for bent caps. For simplicity, Strength III through V, are not shown here as Strength I and II usually governs in most bent cap designs. Furthermore, moments from prestressing, creep, shrinkage, stream pressure, uniform temperature change, temperature gradient, and settlement are not being considered in the examples in this chapter also:

  Strength I

  $$M_{u(HL-93)} = 1.25\left(M_{DC}\right) + 1.5\left(M_{DW}\right) + 1.75\left(M_{HL-93}\right) \tag{5.6.3-1}$$

  Strength II

  $$M_{u(P-15)} = 1.25\left(M_{DC}\right) + 1.5\left(M_{DW}\right) + 1.35(M_{P-15}) \tag{5.6.3-2}$$

  where:

  $M_{u(HL93)}$ = factored moment at the section from HL-93 Vehicle

  $M_{u(P-15)}$ = factored moment the section from the Permit Vehicle

  M_{DC} = unfactored moment at the section from dead load of structural components and nonstructural attachments

  M_{DW} = unfactored moment at the section from dead load of wearing surfaces and utilities

  From the above two limit states, the larger of the two values is the controlling moment. It is possible to have different limit states control at different locations along the bent cap.

- *Calculate Minimum Reinforcement Design Moment, $M_{\min}$*

  The minimum reinforcement requirement ensures that the flexural design of the bent cap provides either enough post-cracking ductility of the member or a modest margin of safety over M_u:

Per AASHTO 5.6.3.3, the minimum factored moment effect is:

$$M_{\min} = \min(M_{cr}, 1.33M_u) \tag{5.6.3-3}$$

Where

M_{cr} = cracking moment (kip-in.)

- *Determine the Factored Moment,* M_u

The maximum value of M_u is:

$$M_u = \max(M_{u(HL-93)}, M_{u(P-15)}, M_{\min}) \tag{5.6.3-4}$$

- *Assume an Initial Value for the Area of Nonprestressed Tension Reinforcement,* A_s

- *Assume the resistance factor,* ϕ

For flexure design, the AASHTO-CA BDS-8 specifies a variable resistance factor. The relationship between ϕ and the steel net tensile strain, ε_t, is provided in the design specifications. The tension-control based resistance factor ϕ is usually assumed.

- *Determine whether the Section is Rectangular or Flanged*

For monolithic integral bent caps, Article 4.6.2.6.5 specifies that the effective flange width overhanging each side of the bent cap web as shown in Figure 5.6.3-1 shall not exceed six times the least slab thickness or one-tenth of the span length.

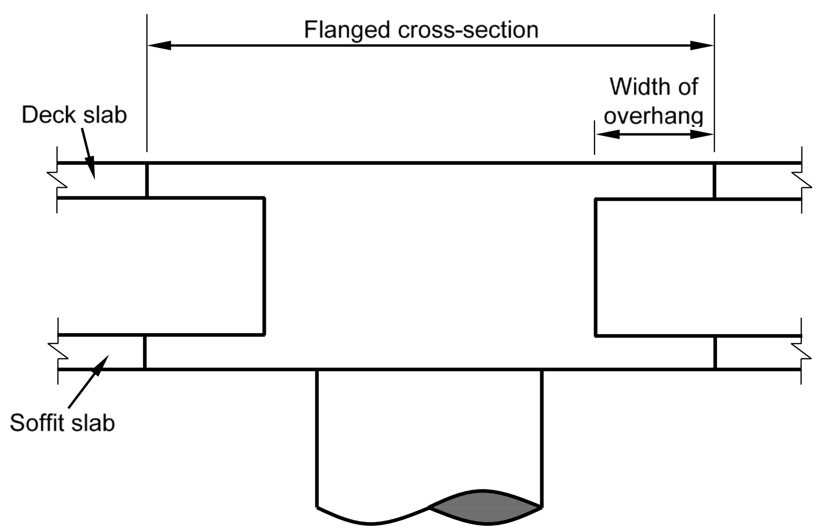


Figure 5.6.3-1 Flanged Cross Section

In the geometrical model shown in Figure 5.6.3-2, the flanged cross-section spans from the centerline of the left exterior girder to the centerline of the right exterior girder at the CG of the bent. The rectangular cross-sections span from the centerline of the exterior girder to the edge of deck. These sections are assumed as rectangular in order to simplify the analytical model.

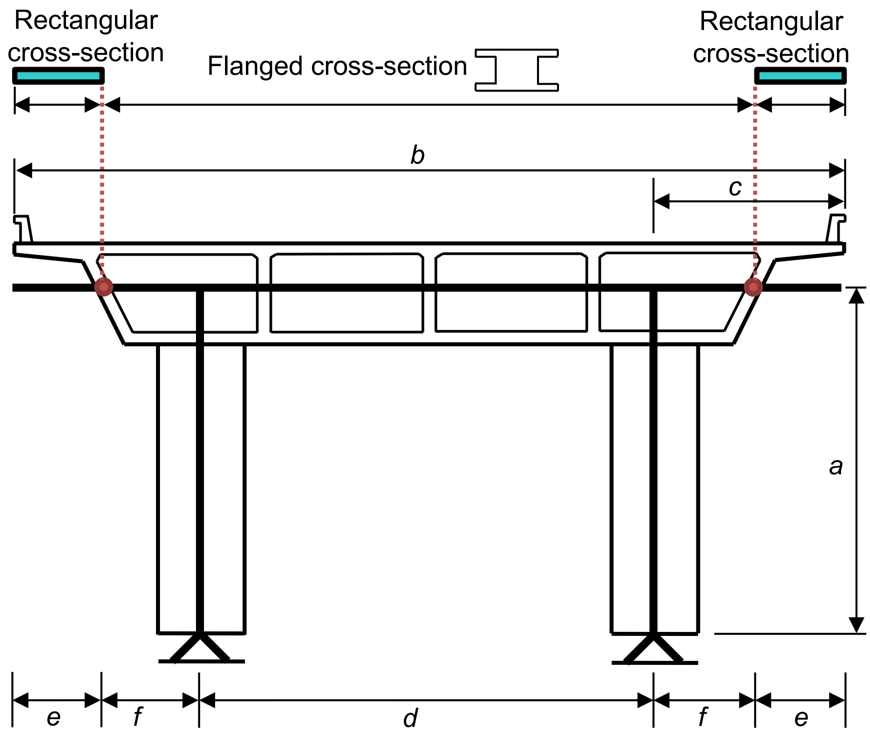


Figure 5.6.3-2 Geometric Model

where:

- *a* = distance from the center of gravity (CG) of the superstructure to the bottom of column = height of the column + depth to CG of the bent cap
- *b* = top width of superstructure
- *c* = distance from the centerline (CL) of the columns to the edge of deck (*e* + *f*)
- *d* = distance between the CL of columns
- *e* = distance from the CL of exterior girder to the edge of deck (EOD)
- *f* = distance from the CL of the column to the CL of the exterior girder

For drop bent caps, the section is rectangular for the full length of the bent cap.

Although the terms "flanged section" and "rectangular section" describe the geometric section, it is important to note that they are not necessarily accurate depictions of the analytical section. A "flanged section" may be analyzed as a "rectangular section." The scenario exists when the depth of the compression zone, i.e, distance from the extreme compression fiber to the neutral axis, c, is less than the thickness of the compression flange, h_f ($c \le h_f$). If the depth of the compression zone is greater than the thickness of the compression flange ($c > h_f$), then the section will exhibit "flanged section" behavior.

- *Calculate Nominal Flexural Resistance M_n*

Nominal flexural resistance shall be in accordance with Article 5.6.3.2 following the flowchart as shown in Figure 5.6.3-3.

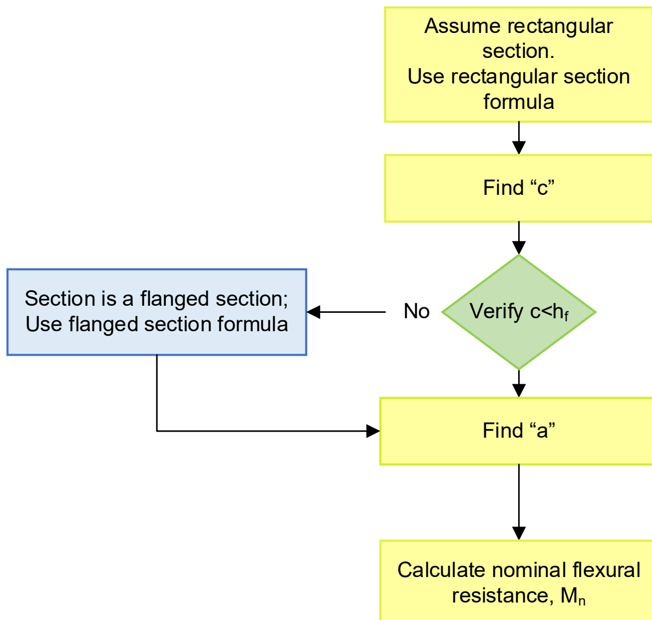


Figure 5.6.3-3 Flexural Resistance Calculation Flowchart

- *Determine the Factored Flexural Resistance, M_r*

The factored flexural resistance is:

$$M_r = \phi M_n \qquad \text{(AASHTO 5.6.3.2.1-1)}$$

The detail discussions of the serviceability check and Fatigue limit state are in Section 5.1.4.3 and 5.1.4.4, respectively in BDP Chapter 5.1.

5.6.3.2 Shear Design

The shear design of bent caps involves:

- Determining the stirrup bar size along the length of the bent cap
- Determining the stirrup spacing along the bent cap

- Checking shear-flexure interaction

The AASHTO-CA BDS-8 shear design method is based on the Modified Compression Field Theory. Contrary to the traditional shear design methodology, it assumes a variable angle truss model instead of the 45º truss analogy, and also accounts for interaction between shear, torsion, flexure, and axial load, as well as residual tension in concrete after cracking, which was neglected in the traditional method of shear design. CA Article 5.7.3.4.2 specifies that shear resistance of all prestressed and nonprestressed sections shall be determined by AASHTO Appendix B5. Section 5.1.5.2 of BDP Chapter 5.1 provides a detailed discussion.

Refer to Section 5.1.5.2.3 of BDP Chapter 5.1 for the longitudinal reinforcement for tension check (Shear-Flexure Interaction)

5.6.3.3 Seismic Design

For seismic design, resistance factor shall be taken as ϕ = 1.0 per CA 5.5.5.1. Refer to BDP Chapter 20.1 for details.

5.6.4 DETAILING CONSIDERATIONS

In addition to the main top and bottom longitudinal steel reinforcement for flexure and vertical stirrups for shear, additional reinforcement is required for side faces and ends for crack control, as well as for construction purposes.

5.6.4.1 Construction Reinforcement

This reinforcement is only needed for box girder construction. Concrete for the box girder is usually placed in two stages. The first stage includes placing the soffit slab and the girder stems. It is commonly known as the “stem and soffit” pour since the deck slab is not included. The second stage consists of constructing the top three to four inches of the stem and the deck slab as shown in Figure 5.6.4-1.

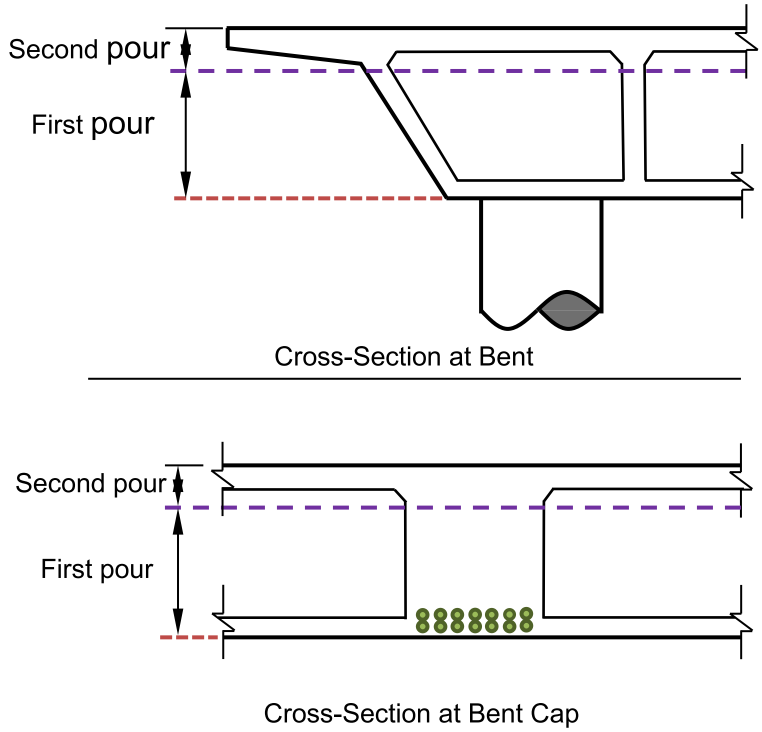


Figure 5.6.4-1 Concrete Pour Stages

The two-stage pouring of concrete results in a construction joint at the bent cap and at the top of the girder stem. At this stage (after first pour), the girders are not stressed, longitudinal bottom steel is in place, and the entire bridge is supported on falsework.

If falsework underwent any settlement or failed unexpectedly due to impact by an errant vehicle, the bent cap will be subjected to a bending moment similar to that shown in Figure 5.6.4-2. As shown, there is negative moment at and near the supports (columns) with no top steel to resist this moment.

As such, it is Caltrans practice to provide additional top longitudinal steel, also commonly referred to as construction reinforcement, underneath the construction joint for the potential loss or settlement of the falsework supporting the bridge. A 10-ft length of the exterior stem girder on each side of the bent cap is assumed to act as dead load weight in addition to the bent cap own weight. Detailed weight calculations and design consideration for construction reinforcement are shown in Section 5.6.6.3, Integral Bent Cap Example. See Figure 5.6.4-3.

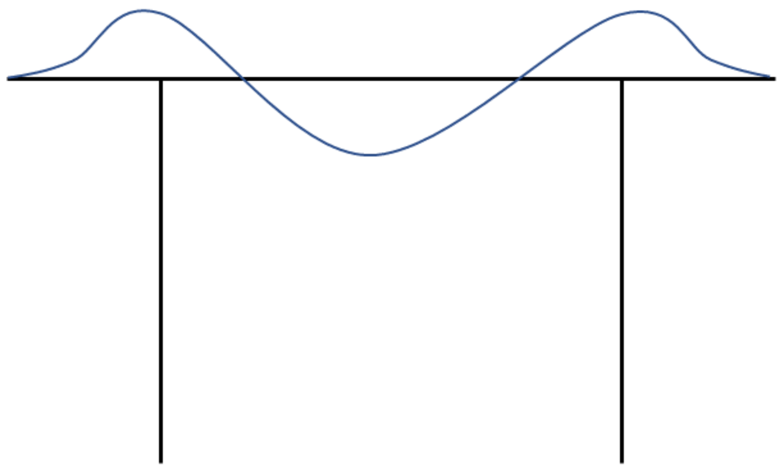


Figure 5.6.4-2 Potential Moment Diagram after First Pour

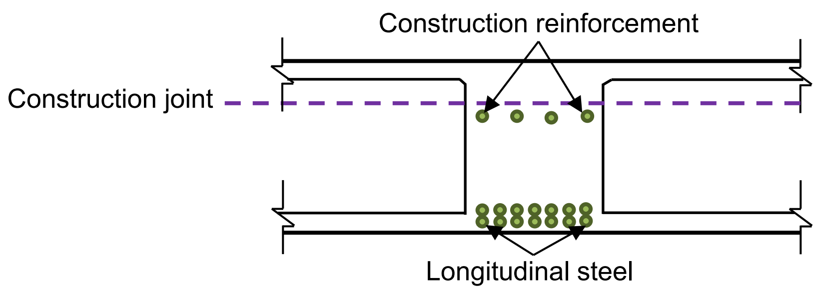


Figure 5.6.4-3 Construction Reinforcement

5.6.4.2 Side Face Reinforcement

Caltrans SDC 7.4.5.2-3 and 7.4.5.3-4 require that horizontal side face reinforcement A_s^{sf} shall be greater or equal to 10 percent of the maximum amount of longitudinal reinforcement of the bent cap. The maximum amount of longitudinal reinforcement will come from either the top or bottom steel. The side face reinforcement, shown in Figure 5.6.4-4, is placed along the two vertical faces of the bent cap and shall have a maximum spacing of 12 inches.

It is permissible to include the construction reinforcement to satisfy part of this requirement.

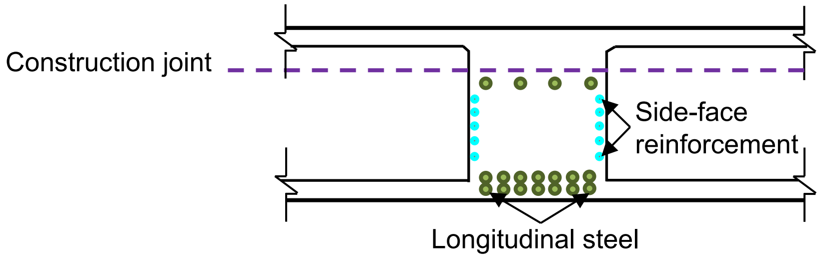


Figure 5.6.4-4 Side Face Reinforcement

5.6.4.3 End Reinforcement

The end reinforcement is provided along the end face of the bent cap as a crack control measure. There are two basic types of end reinforcement, Z-bars and U-bars as shown in Figure 5.6.4-5.

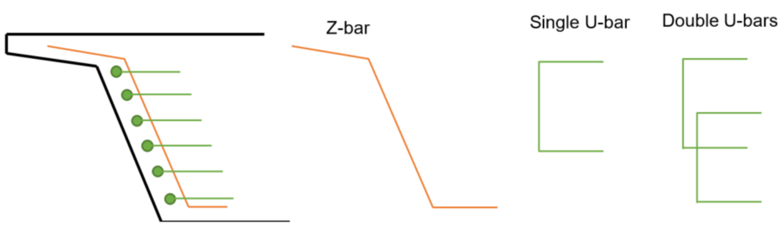


Figure 5.6.4-5 End Reinforcement in the Bent Cap

The U-bars (horizontal plane) are typically designed using the shear friction concept to resist the dead and live load of the exterior girder. The number of U-bars per set depends on the bent cap width. From past experiences, the double U-bars are usually used for the bent cap width larger than 7 ft.

5.6.4.4 Other Detailing Considerations (Skew)

If the skew angle of the bent is 20° or less, the deck and soffit slab reinforcement are placed parallel to the centerline of bent cap. This bar reinforcement configuration allows the bent cap longitudinal reinforcement to be placed as far from the extreme compression fiber as possible to optimize flexural capacity. When the bent cap skew angle is greater than 20°, the deck and soffit slab reinforcement are typically placed perpendicular to the centerline of the bridge and the bent cap top longitudinal steel must be placed below the deck reinforcement or above the soffit reinforcement. Details of bent cap reinforcement for different skew angles shall be in accordance with SDC 7.4.5.

Figure 5.6.4-6 shows a typical bent cap reinforcement details for a bent cap with the skew angle less than 20°. A dropped deck section may be required if main cap bars are bundled vertically. Distribution bars and bottom transverse bars may have to be terminated farther from the bent cap than three inches (standard) to allow vertical clearance for main bent bars.

- Slab reinforcement parallel to skew
- Bent cap reinforcement as high as possible

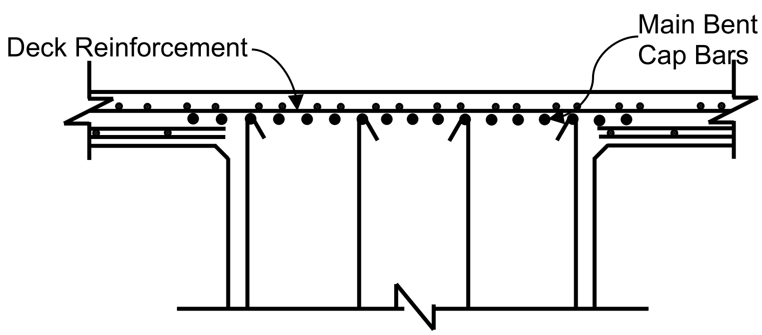


Figure 5.6.4-6 Typical Bent Cap Reinforcement (Skew ≤ 20°)

5.6.5 DESIGN PROCEDURE

Figure 5.6.5-1 shows the flowchart of analysis and design steps of a typical bent cap.

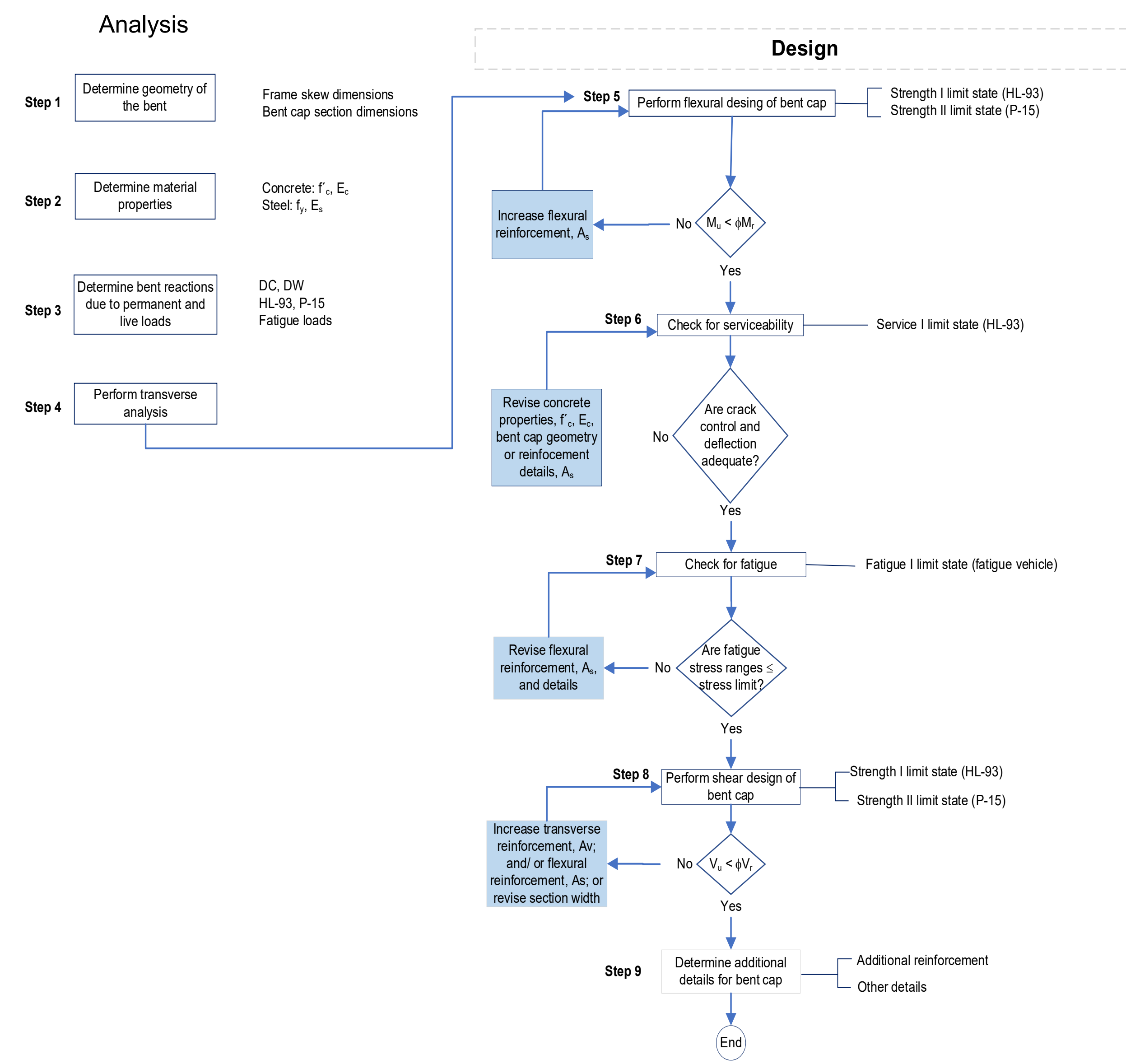


Figure 5.6.5-1 Bent Cap Analysis and Design Flowchart

5.6.6 INTEGRAL BENT CAP DESIGN EXAMPLE

The information contained in this section should not be used to replace reading the design specifications. There are often several ways to solve a design problem. It is recommended that prior to applying any formula or procedure contained within this section, the designer should read the appropriate Articles of AASHTO-CA BDS-8 and the SDC to be certain that the described formula or procedure is appropriate for use.

It should be noted that the example does not constitute a complete bent cap design. Only selected work has been done to illustrate design methods. For example, tension steel has not been designed for every span of the bent cap as it would be done for an actual bent cap design. Additionally, there are other design considerations not considered in this example. For instance, seismic design is not included. It is hoped, however, that the example will provide a good foundation for the design of bent caps.

5.6.6.1 Bent Cap Data

An integral bent cap is supported by two six feet diameter concrete columns as shown in Figure 5.6.6-1. The box girder has four cells of depth 6.75 ft with a skew angle of 20°.

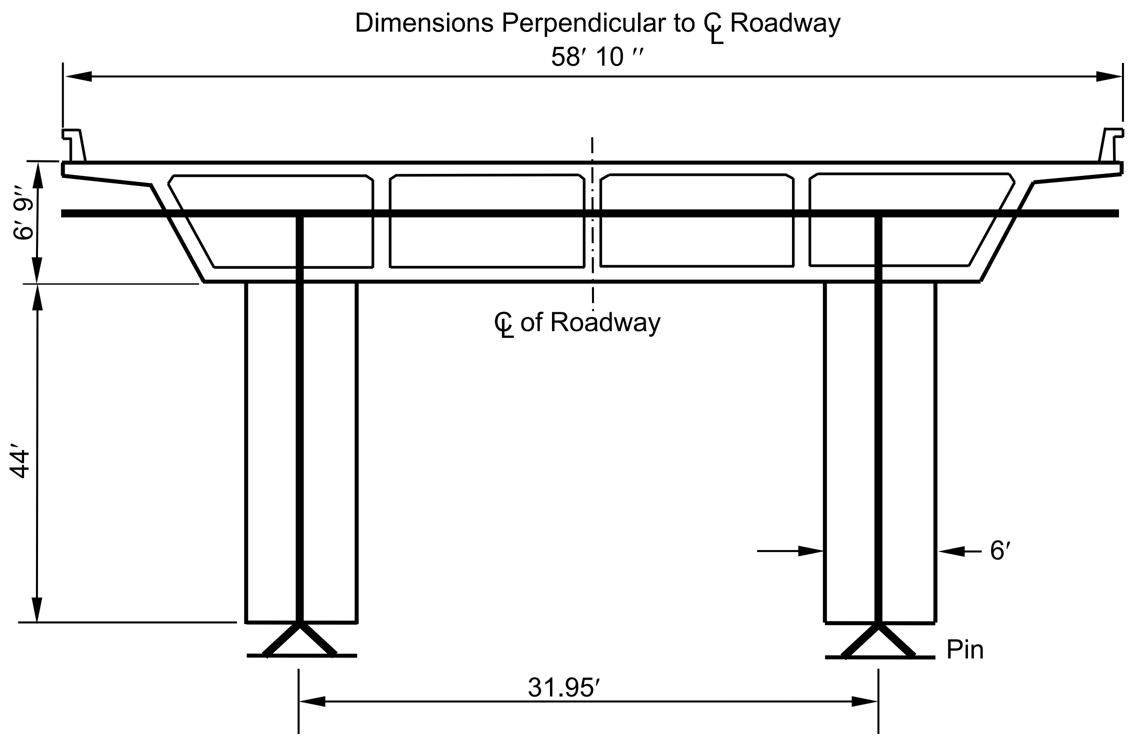


Figure 5.6.6-1 Two-Column Bent

Note: These dimensions are perpendicular to the centerline (CL) of the roadway and are not actual dimensions of the bent since the bent is skewed at an angle to the roadway.

5.6.6.2 Design Requirements

Perform the structural analysis, flexural, and shear design for the bent cap as shown in Figure 5.6.6-1 in accordance with the AASHTO-CA BDS-8.

5.6.6.3 Design

5.6.6.3.1 Step 1: Determine Geometry of Bent

Bent dimensions along the skew are shown in Figure 5.6.6-2. Ranges of bent cross section types are shown in Figure 5.6.6-3.

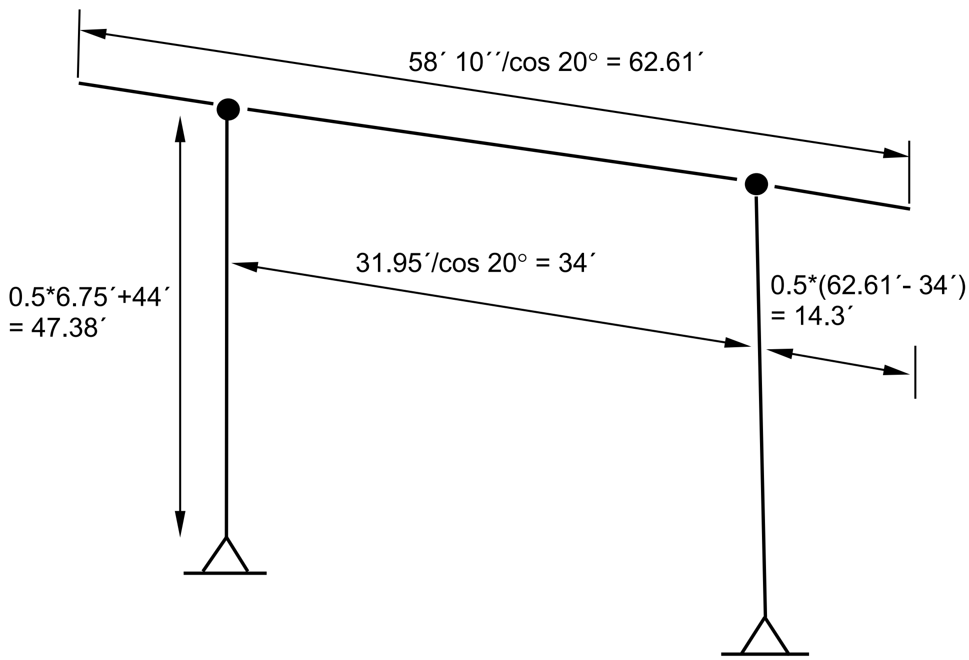


Figure 5.6.6-2 Bent Dimensions along the Skew

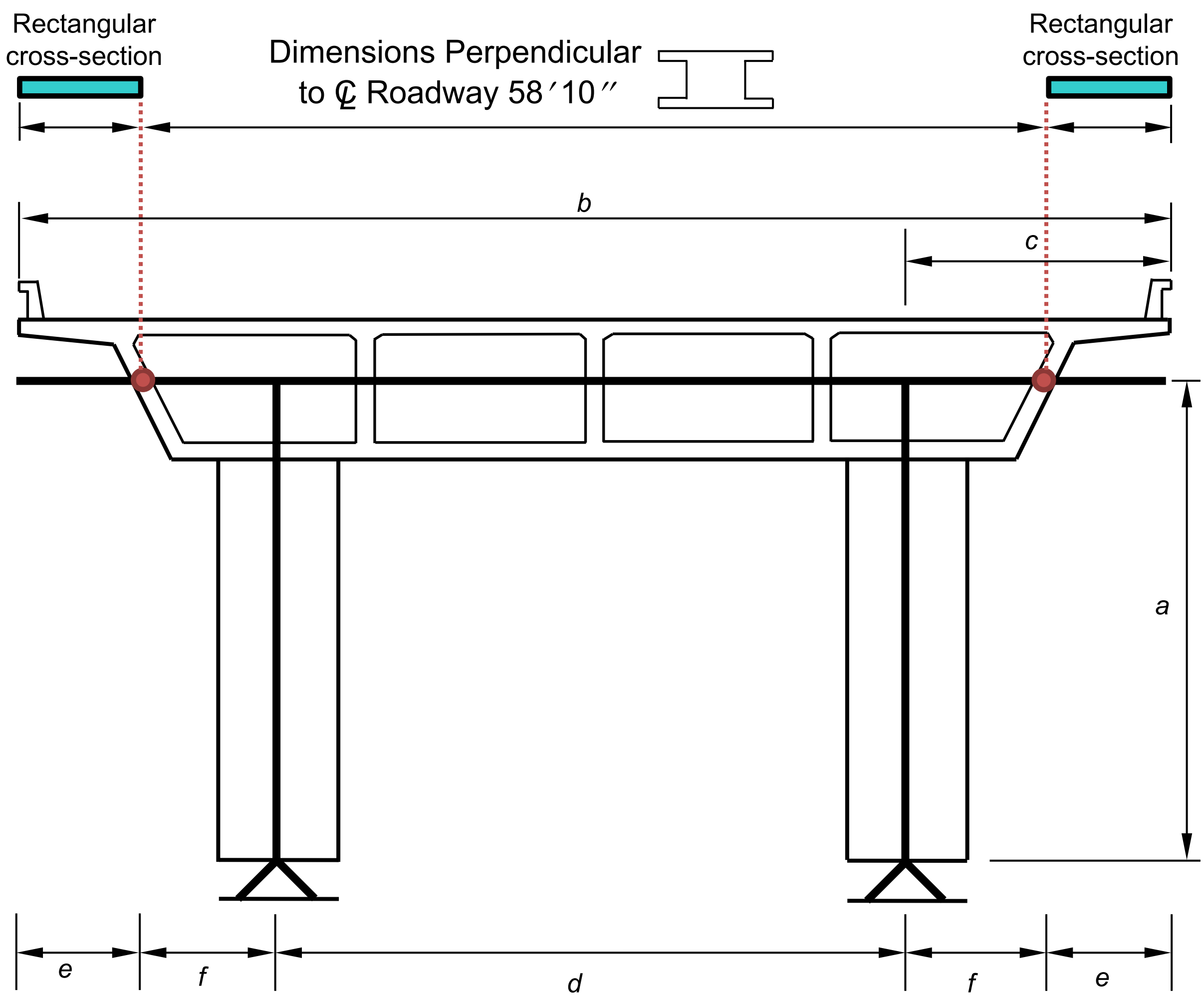


Figure 5.6.6-3 Bent Cross Section Types

where :

a = distance from the centroid of gravity of the superstructure to the bottom of column

b = top width of superstructure

c = distance from the CL of the columns to the edge of deck (e + f)

d = distance between the CL of columns

e = distance from the CL of exterior girder to the edge of deck (EOD)

f = distance from the CL of the column to the CL of the exterior girder

For the example bridge, assume that CG of the bent cap is at mid-depth of the cap beam.

- *Rectangular Cross Section*

The rectangular cross section spans from the CL of the exterior girder to the EOD. These sections are assumed as rectangular in order to simplify the stick model.

The dimension *e* in the geometric model is the distance from the CL of exterior girder to the EOD along the skew, as shown in Figure 5.6.6-4.

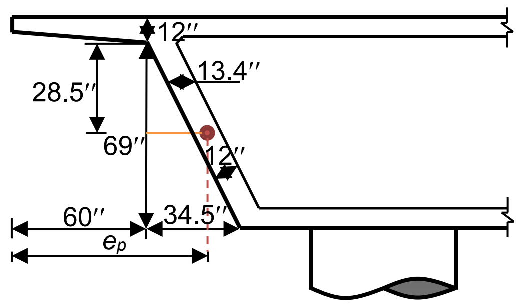


Figure 5.6.6-4 Rectangular Cross Section

$$e_p = \frac{1}{2}(13.4) + \left[\frac{28.5}{69}(34.5)\right] + 60 = 81 \text{ in}$$

$$e \text{ (along skew)} = \frac{81}{\cos(20°)} = 86.2 \text{ in.} = 7.18 \text{ ft}$$

$$f = c - e = 14.3 - 7.18 = 7.12 \text{ ft}$$

- *Flanged Cross Section*

Range of flanged cross section is shown in Figure 5.6.6-5.

Minimum bent cap width = column width + 24 in. (SDC 7.4.3-1)

Width of bent cap, B_{cap} = 6 + 2 = 8 ft

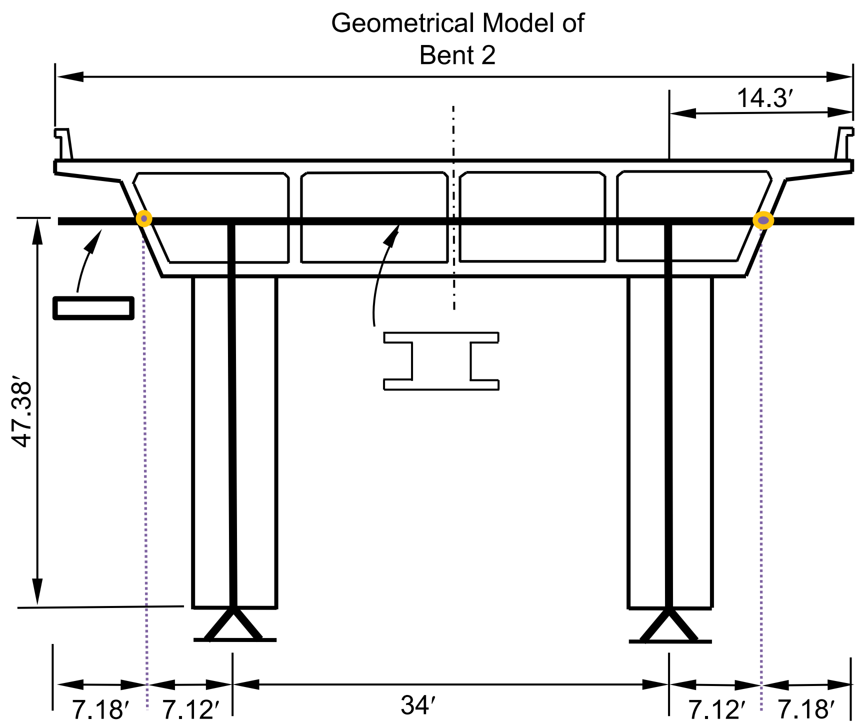


Figure 5.6.6-5 Range of Flanged Cross Section

Effective bent cap flange width = width of bent cap + 2 (width of overhang at each side), as shown in Figure 5.6.6-6.

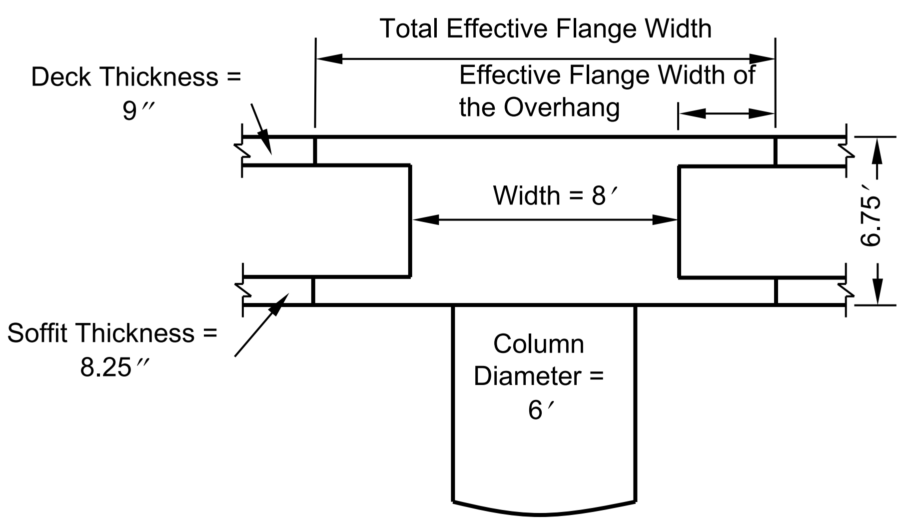


Figure 5.6.6-6 Effective Flange Width

Per AASHTO 4.6.2.6.5, effective flange width overhanging each side for the section between columns for negative moment region is:

$$\text{Least of}\begin{bmatrix} 6\ (\text{least soffit slab thickness}) = 6(8.25) = 49.5\text{in.} \\ 0.1\ (\text{span length of the bent cap}) = 0.1(34) = 3.4\ \text{ft} = 40.8\ \text{in.} \end{bmatrix}$$

Effective flange width overhanging each side for the section between columns for the positive moment region is:

$$\text{Least of}\begin{bmatrix} 6\ (\text{least deck slab thickness}) = 6(9.0) = 54.0\ \text{in.} \\ 0.1\ (\text{span length of the bent cap}) = 0.1(34) = 3.4\ \text{ft} = 40.8\ \text{in.} \end{bmatrix}$$

It is seen that for both positive and negative moment regions, total effective flange width, b_e = 8 + 3.4 + 3.4 = 14.8 ft = 177.6 in.

5.6.6.3.2 Step 2: Determine Material Property

A706 Steel reinforcement with f_y = 60 ksi and E_s = 29,000 ksi, and concrete with f_c' = 4 ksi and E_c = 3,645 ksi, are used in this design example.

5.6.6.3.3 Step 3: Determine Bent Reactions due to Permanent and Live Loads

Bent reactions due to permanent and live load are calculated as follows.

- *Reactions due to Permanent Loads*

  The permanent loads are comprised of:

  Dead load of structural components and non-structural attachments (*DC*)

  Dead load of wearing surfaces and utilities (*DW*)

  Self-weight of bent cap

  The unfactored bent reactions *for DC* and *DW* obtained from a longitudinal analysis are shown in Table 5.6.6-1.

- *Self-Weight of the Bent Cap*

  The self-weight of the bent cap is modeled as a uniformly distributed load as shown in Figure 5.6.6-7.

  Self-weight of bent cap = (cross-sectional area of the bent cap solid section) × (average width of bent cap) × (unit weight of concrete)

  Average length = 7.12 + 34 + 7.12 = 48.24 ft

  Box cell area = 217.9 ft^2

  Bent cap self-weigh = (217.90) (8) (0.15)= 261.48 kip

  Self-weight modeled as uniform load = 261.48 / 48.24 = 5.42 kip/ft

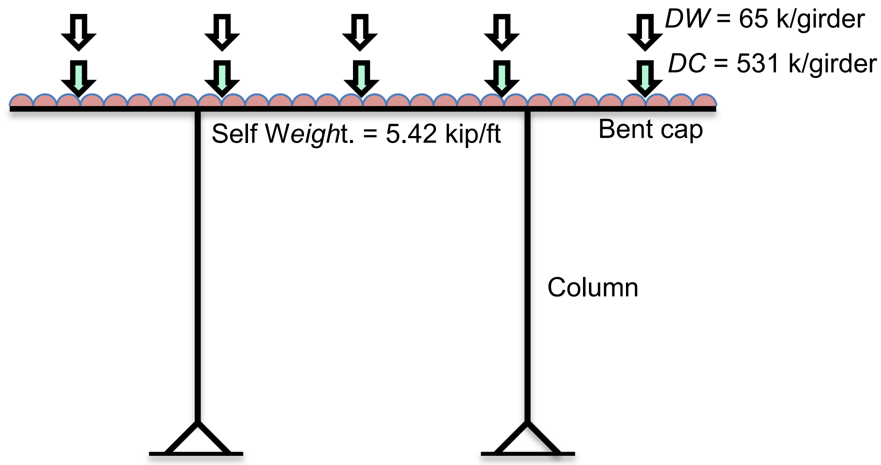


Figure 5.6.6-7 Permanent Load on Bent Cap

Note: While calculating the self-weight of the bent cap, be careful not to include the portion of the deck, soffit slab, and girder thicknesses at the bent cap if they have already been included in the longitudinal analysis of the bridge.

Table 5.6.6-1 Unfactored Reactions due to *DC*, *DW,* HL-93, P-15, and Fatigue Vehicles

	Bent Reactions (kip)	Dynamic Allowance Factor	Final Bent Reactions (kip)
DC	2,651.50	NA	2,651.50
DW	325.00	NA	325.00
Self-Weight	261.48	NA	261.48
HL-93 Vehicle Truck Lane	114.82 99.34	1.33 1.00	252.00
Permit Vehicle	360.77	1.25	451.00
Fatigue Vehicle	70.66	1.15	81.25

For both *DC* and *DW*, these bent reactions obtained from the longitudinal analysis are modeled as concentrated loads acting at the CL of each girder framing into the bent cap.

- Reaction due to *DC* on the bent cap for each girder

$$= \frac{2651.5}{5} = 530.3 \text{ kip}$$

- Reaction due to *DW* on the bent cap for each girder

$$= \frac{325.0}{5} = 65.0 \text{ kip}$$

- *Live Load as Two Wheels*

  Vehicular live loads as two wheels are applied on the bent caps as shown in Figure 5.6.6-8.

  HL-93 vehicle: $\frac{252.0}{2} = 126.0$ kip

  Permit vehicle: $\frac{451.0}{2} = 225.5 \text{ kip} \approx 226.0$ kip

  Fatigue vehicle: $\frac{81.25}{2} = 40.6 \text{ kip} \approx 41.0$ kip

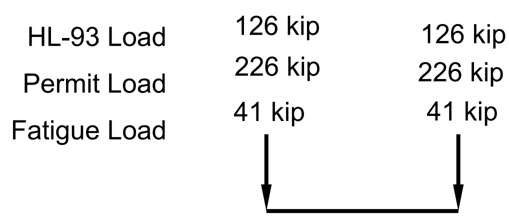


Figure 5.6.6-8 Bent Reaction due to Live Loads

The HL-93 vehicle live loads can be one, two, three, or four lanes.

Figure 5.6.6-9 shows different HL-93 truck placement for transverse analysis using CSiBridge. Figure 5.6.6-10 shows placement of two HL-93 lanes.

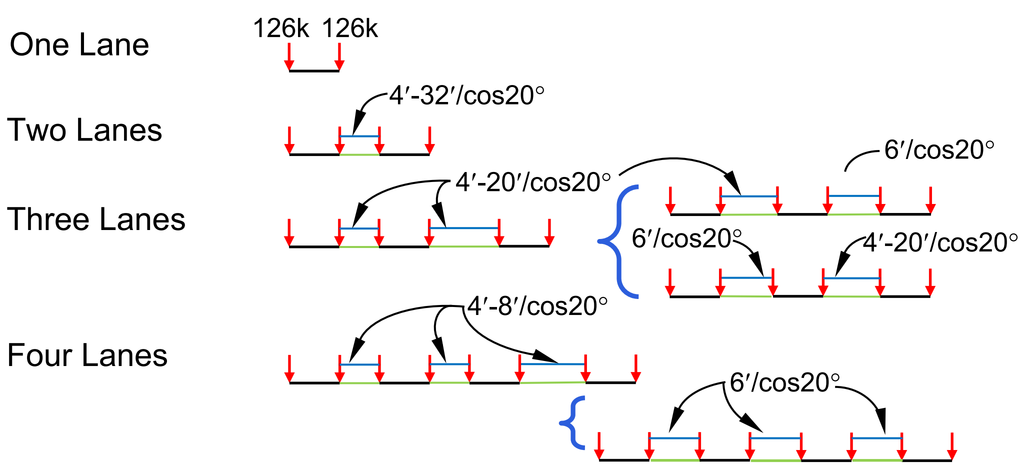


Figure 5.6.6-9 Possible Scenarios for Placement of Live Load

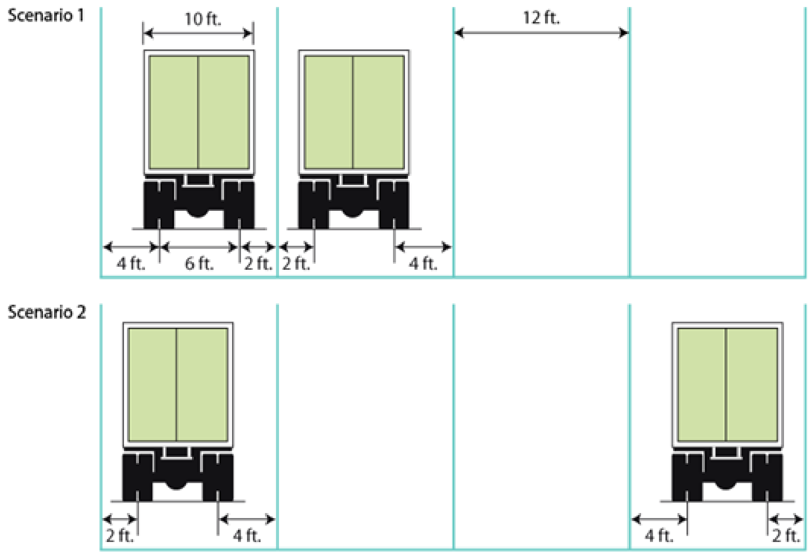


Figure 5.6.6-10 Two Scenarios for Two Truck Case

Permit vehicle live load may be only one or two lanes as shown in Figure 5.6.6-11.

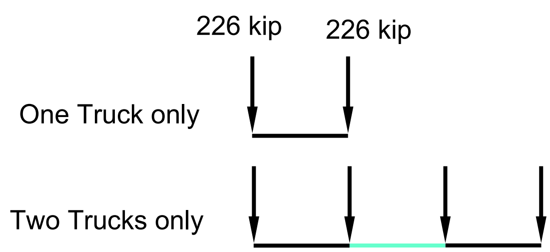


Figure 5.6.6-11 Permit Trucks

Fatigue vehicle live load is only one truck as shown in Figure 5.6.6-12.

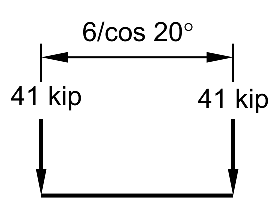


Figure 5.6.6-12 Fatigue Truck

5.6.6.3.4 Step 4: Perform Transverse Analysis

The goal of the transverse analysis is to obtain force effect envelopes of all possible live load cases. The example bridge has four design lanes. Each of the different live load trucks can be placed in these design lanes. The number and placement of these trucks depends on the type of load:

- HL-93 vehicle
- Permit truck
- Fatigue truck

Since there may be many live load cases to consider, computer software is usually used to determine:

- Maximum moment (negative or positive) and associated shear
- Maximum shear and associated moment

Available computer programs for transverse analysis are:

- CSiBridge
- VBENT
- CTBridge

For this example bridge, results are obtained by performing transverse analysis with CSiBrdge.

The results from the transverse analysis are obtained at every tenth point or at selected points. These results are shown separately for *DC*, *DW*, HL-93, permit, and fatigue loads. Tables 5.6.6-2 and 5.6.6-3 list the controlling unfactored moments.

- *Calculate Factored Moments*

  Table 5.6.6-2 shows unfactored controlling moments including impact for the bent cap, obtained from the transverse analysis of the bridge using CSiBridge.

Table 5.6.6-2 Unfactored Moments

Load and Moment	Moment at Midspan (kip-ft)	Moment at Face of Column (kip-ft)
DC, M_{DC}	3,377	-1,760
DW, M_{DW}	339	-217
HL-93 Vehicle, $M_{HL\text{-}93}$	2,683	-1,859
Permit Vehicle, $M_{P\text{-}15}$	4,571	-3,336

Calculate the factored moment at the midspan as follows:

- Strength I

$$\begin{aligned} M_u &= 1.25M_{DC} + 1.5M_{DW} + 1.75M_{HL-93} \\ &= (1.25)(3{,}377) + (1.5)(339) + (1.75)(2{,}683) = 9{,}425 \text{ kip-ft} \end{aligned}$$

- Strength II

$$\begin{aligned} M_u &= 1.25M_{DC} + 1.5M_{DW} + 1.35M_{P15} \\ &= (1.25)(3{,}377) + (1.5)(339) + (1.35)(4{,}571) = 10{,}901 \text{ kip-ft} \end{aligned}$$

For the example bent cap, f_{cpe} = 0. The bent cap is designed for the monolithic section to resist all loads. Substitute S_{nc} for S_c and cracking moment is calculated by:

$$M_{cr} = \gamma_3(\gamma_1 f_r)S_c \qquad \text{(AASHTO 5.6.3.3-1)}$$

Where:

$$f_r = 0.24\lambda\sqrt{f_c'} = (0.24)(1.0)\sqrt{4} = 0.48 \text{ ksi} \qquad \text{(Article 5.4.2.6)}$$

$\gamma_1 = 1.60$ (for concrete structure, AASHTO 5.6.3.3)

$\gamma_3 = 0.75$ (for A706, Grade 60 reinforcement, AASHTO 5.6.3.3)

I_g (flanged section) = 280.5 ft^4; Y_t = 40.30 in.; Y_b = 40.70 in.

$$S_c = \frac{I_g}{Y_b} = \frac{280.5}{(40.70/12)} = 82.70 \text{ ft}^3$$

$$M_{cr} = (0.75)\left[(1.6)(0.48)(12)^2\right](82.7) = 6{,}860 \text{ kip-ft}$$

$$\begin{aligned} M_{u(\min)} &= \text{Lesser of} \begin{Bmatrix} 1.0M_{cr} \\ 1.33M_u \end{Bmatrix} \\ &= \text{Lesser of} \begin{Bmatrix} 1.0(6{,}860) = 6{,}860 \text{ kip-ft} \\ 1.33(10{,}901) = 14{,}498 \text{ kip-ft} \end{Bmatrix} = 6{,}860 \text{ kip-ft} \end{aligned} \qquad (5.6.3\text{-}3)$$

Therefore, the controlling factored moment at midspan is:

M_u = 10,901 kip-ft

Calculate the factored moment at the face of column as follows:

- Strength I

$$M_u = 1.25M_{DC} + 1.5M_{DW} + 1.75M_{HL-93}$$
$$= (1.25)(-1{,}760) + (1.5)(-217) + (1.75)(-1{,}859) = -5{,}779 \text{ kip-ft}$$

- Strength II

$$M_u = 1.25M_{DC} + 1.5M_{DW} + 1.35M_{P-15}$$
$$= (1.25)(-1{,}760) + (1.5) - 217) + (1.35)(-3{,}336) = -7{,}029 \text{ kip-ft}$$

For the example bent cap, f_{cpe} = 0. Also, the bent cap is designed for the monolithic section to resist all loads. Substitute S_{nc} for S_c and cracking moment is calculated by:

$$M_{cr} = \gamma_3(\gamma_1 f_r)S_c \qquad \text{(AASHTO 5.6.3.3-1)}$$

Where:

$$f_r = 0.24\lambda\sqrt{f_c'} = (0.24)(1.0)\sqrt{4} = 0.48 \text{ ksi} \qquad \text{(AASHTO 5.4.2.6)}$$

$\gamma_1 = 1.60$ (for concrete structure, AASHTO 5.6.3.3)

$\gamma_3 = 0.75$ (for A706, Grade 60 reinforcement, AASHTO 5.6.3.3)

I_g (flanged section) = 280.5 ft^4; Y_t = 40.30 in.; Y_b = 40.70 in.

$$S_c = \frac{I_g}{Y_b} = \frac{(280.5)(12)}{(40.30)} = 83.52 \text{ ft}^3$$

$$M_{cr} = (0.75)\left[(1.6)(0.48)(12)^2\right](83.52) = 6{,}928 \text{ kip-ft}$$

$$M_{u(\min)} = \text{Lesser of } \begin{Bmatrix} 1.0M_{cr} \\ 1.33M_u \end{Bmatrix}$$
$$= \text{Lesser of } \begin{Bmatrix} 1.0(6{,}928) = 6{,}928 \text{ kip-ft} \\ 1.33(7{,}029) = 9{,}349 \text{ kip-ft} \end{Bmatrix} = 6{,}928 \text{ kip-ft} \qquad (5.6.3\text{-}3)$$

Therefore, the controlling factored negative moment at the face of column is:

$$|M_u| = 7{,}029 \text{ kip-ft}$$

Calculate the factored Fatigue I limit state moment at the midspan using the unfactored fatigue moments at midspan as listed in Table 5.6.5-3.

Table 5.6.6-3 Unfactored Fatigue Load Moments

Load	Max. and Min. Moment at Midspan (kip-ft)		Max. and Min. Moment at Face of Column (kip-ft)	
	+Positive	-Negative	+Positive	-Negative
DC	3,377	0	0	-1,760
DW	339	0	0	-217
Fatigue Vehicle I	789	-264	144	-504

For Fatigue I load combination, a load factor of 1.75 shall be used.

$$M_{u(max)} = 3{,}377 + 339 + 1.75(789) = 5{,}097 \text{ kip-ft}$$

$$M_{u(min)} = 3{,}377 + 339 + 1.75(-264) = 3{,}254 \text{ kip-ft}$$

- *Calculate Factored Shear*

Table 5.6.6-4 lists unfactored shears including impact from CSiBridge analysis:

Table 5.6.6-4 Unfactored Shear at Face of Support

Load	Max. Shear (kip)	Assoc. Moment (kip-ft)	Max. Moment (kip-ft)	Assoc. Shear (kip)
DC	**-888**	**-1760**	**-1760**	**-888**
DW	**-98**	**-217**	**-217**	**-98**
Design Vehicle	**-338**	**588**	**-1860**	**-66**
Permit Vehicle	**-607**	**1054**	**-3335**	**-119**

Strength I: $V_{u(max)}$

$$V_u = 1.25(-888) + 1.5(-98) + 1.75(-338) = -1{,}849 \text{ kip}$$

$$M_{u(assoc)} = 1.25(-1{,}760) + 1.5(-217) + 1.75(558) = -1{,}549 \text{ kip-ft}$$

Strength II: $V_{u(max)}$

$$V_u = 1.25(-888) + 1.5(-98) + 1.35(-607) = -2{,}076 \text{ kip}$$

$$M_{u(assoc)} = 1.25(-1{,}760) + 1.5(-217) + 1.35(1{,}054) = -1{,}102 \text{ kip-ft}$$

To determine the tension in the longitudinal steel (the shear-flexure interaction), check at every location, usually at the 10th point of the span and at concentrated load:

1. Maximum shear and associated moments
2. Maximum positive moments and associated shear

3. Maximum negative moments and associated shear

Longitudinal reinforcement at the first interior girder locations is checked in this example.

Table 5.6.6-5 lists unfactored shears including impact from CSiBridge analysis at the location of the first interior girder:

Table 5.6.6-5 Unfactored Shears at Location of First Interior Girder

Load	Max. Shear (kip)	Assoc. Moment (kip-ft)	Max. Moment (kip-ft)	Assoc. Shear (kip)
DC	**-878**	**-373**	**-373**	**-878**
DW	**-98**	**-64**	**-64**	**-98**
Design vehicle	**-315**	**990**	**-1756**	**-66**
Permit vehicle	**-565**	**1,776**	**-3,149**	**-119**

- Strength I

Maximum shear and associated moments:

$$V_u = 1.25(-878) + 1.5(-98) + 1.75(-315) = -1,796 \text{ kip}$$

$$M_{u(assoc)} = 1.25(-373) + 1.5(-64) + 1.75(990) = 1,170 \text{ kip-ft}$$

Maximum moment and associated shear:

$$V_{u(assoc)} = 1.25(-878) + 1.5(-98) + 1.75(-66) = -1,360 \text{ kip}$$

$$M_{u(max)} = 1.25(-373) + 1.5(-64) + 1.75(-1,756) = -3,635 \text{ kip-ft}$$

- Strength II

Maximum shear and associated moments:

$$V_u = 1.25(-878) + 1.5(-98) + 1.35(-565) = \underline{-2,007 \text{ kip}}$$

$$M_{u(assoc)} = 1.25(-373) + 1.5(-64) + 1.35(1,776) = \underline{1,835 \text{ kip-ft}}$$

Maximum moment and associate shear:

$$V_{u(assoc)} = 1.25(-878) + 1.5(-98) + 1.35(-119) = \underline{-1,405 \text{ kip}}$$

$$M_{u(max)} = 1.25(-373) + 1.5(-64) + 1.35(-3,149) = \underline{-4,813 \text{ kip-ft}}$$

Check case of maximum shear and associated moment:

$$V_{u(max)} = -2,007 \text{ kip}$$

$$M_{u(assoc)} = 1,835 \text{ kip-ft}$$

Note: The detailed process of analyzing the bent using CSiBridge is not covered under this chapter.

5.6.6.3.5 Step 5: Perform Flexural Design

The design equation is as follows:

$$M_u \le M_r = \phi M_n \tag{5.6.6-1}$$

- *Check Positive Moment at Midspan*

  For the bent section without prestressing steel and with neglecting the effect of compression reinforcement, the M_n equation, AASHTO Eq. (5.6.3.2.2-1) reduces to:

$$M_n = A_s \mathrm{f}_s \left(d_s - \frac{a}{2} \right) + 0.85 \mathrm{f}_c' \left(b - b_w \right) h_\mathrm{f} \left(\frac{a}{2} - \frac{h_\mathrm{f}}{2} \right) \tag{5.6.6-2}$$

  Assuming the neutral axis lies in the compression flange (rectangular section behavior), nominal flexural resistance M_n is calculated by:

$$M_n = A_s f_s \left(d_s - \frac{a}{2} \right) \tag{5.6.6-3}$$

  where:

  $M_n =$ nominal flexural resistance (kip-in.)

  $f_s = 60$ ksi

  Assume vertical bundle of #11s and clearance to edge of bundled bars from soffit = 5.7 in.

  $d_s = 81 - (5.70 + 1.63) = 73.67$ in.

  $b = b_e = 177.6$ in.

  $b_w = 96$ in.

  $h_f = 9$ in.

  $a = c\beta_1$

  where:

  $\beta_1 = 0.85$ (Article 5.6.2.2)

$$a = \left(\frac{A_s f_s}{0.85 f_c' b} \right)$$

$$c = \left(\frac{A_s f_s}{0.85 f_c' \beta_1 b} \right)$$

Assume ϕ = 0.9, rearrange and substitute design parameters into Eq. (5.6.6-1) to obtain:

$$\frac{M_u}{0.9} \le A_s f_s \left[d_s - \left(\frac{A_s f_s}{0.85 f'_c b} \right) \left(\frac{1}{2} \right) \right] \tag{5.6.6-4}$$

$$\frac{10{,}901(12)}{0.9} \le A_s(60)\left[73.67 - \left(\frac{A_s(60)}{0.85(4)(177.6)} \right) \left(\frac{1}{2} \right) \right]$$

Solving for A_s for positive moment region

Required $A_s \ge 28.65$ in.^{2}

Provide 22- #11 as bottom reinforcement: A_s = 34.32 in.^{2}

$$c = \left(\frac{A_s f_s}{0.85 f'_c \beta_1 b} \right) = \frac{34.32(60)}{(0.85)(4)(0.85)(177.6)} = 4.01 \text{ in.} < 9 \text{ in.}$$

Assumption of the neutral axis within in the flange is correct.

$$M_r = \phi M_n = 0.9(A_s f_y)\left(d_s - \frac{c\beta_1}{2} \right)$$

$$= 0.9(34.32)(60)\left(73.67 - \frac{(4.01)(0.85)}{2} \right)\left(\frac{1}{12} \right)$$

$$= 11{,}114 \text{ kip-ft} > M_u = 10{,}901 \text{ kip-ft} \qquad \text{OK}$$

Strain diagram is shown in Figure 5.6.6-13:

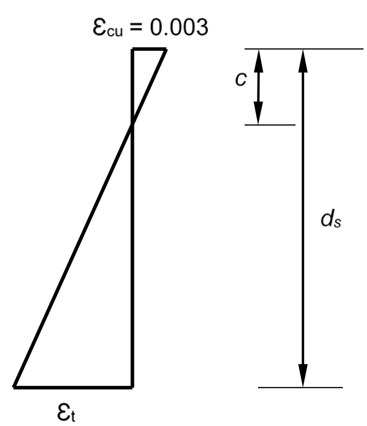


Figure 5.6.6-13 Strain Diagram

$$\left(\frac{\varepsilon_{cu}}{c}\right)=\left(\frac{\varepsilon_t}{d_s-c}\right) \tag{5.6.6-5}$$

$$\left(\frac{0.003}{4.01}\right)=\left(\frac{\varepsilon_t}{73.67-4.01}\right)$$

We obtain $\varepsilon_t = 0.052 > 0.005$. Therefore, assumption of ϕ = 0.90 is correct.

- *Check Negative Moment at the Face of Column*

Assuming the neutral axis is within the compression flange (rectangular section behavior), nominal flexural resistance M_n is calculated by:

$$M_n = A_s f_s\left(d_s-\frac{a}{2}\right) \tag{5.6.6-3}$$

where:

$f_s = 60$ ksi

$b_w = 96$ in.

$b = b_e = 177.6$ in.

Assume vertical bundle of #11s and clearance to edge of bundled bars from soffit = 5.0 in.

$d_s = 81-(5.0+1.63)=74.37$ in.

$a = c\beta_1$

Where $\beta_1 = 0.85$ (Article 5.6.2.2)

Assume ϕ = 0.9, rearrange and substitute design parameters into Eq. (5.6.6.1) to obtain:

$$\frac{M_u}{0.9}\le A_s f_y\left(d_s-\left(\frac{A_s f_y}{0.85 f'_c b}\right)\frac{1}{2}\right) \tag{5.6.6-4}$$

$$\frac{7{,}029(12)}{0.9}\le A_s(60)\left(74.37-\left(\frac{A_s(60)}{0.85(4)(177.6)}\right)\left(\frac{1}{2}\right)\right)$$

Solving for A_s for negative moment at the face of the support:

Required $A_s \ge 21.31\ \text{in.}^2$

Provide 14 - #11 as top reinforcement ($A_s = 21.84$ in.^{2})

$$c = \left(\frac{A_s f_s}{0.85 f_c' \beta_1 b}\right) = \frac{21.84(60)}{(0.85)(4)(0.85)(177.6)} = 2.55 \text{ in.} < 8.25 \text{ in.}$$

Assumption of the neutral axis within the flange is correct

$$M_r = \phi M_n = 0.9\left(A_s f_y\right)\left(d_s - \frac{c\beta_1}{2}\right)$$

$$= 0.9(21.84)(60)\left(74.37 - \frac{2.55(0.85)}{2}\right)$$

$$= 86{,}431 \text{kip-in.}$$

$$= 7{,}203 \text{ kip-ft} > \left|M_u\right| = 7{,}029 \text{ kip-ft} \qquad \text{O.K.}$$

Strain in steel:

$$\left(\frac{\varepsilon_{cu}}{c}\right) = \left(\frac{\varepsilon_t}{d_s - c}\right)$$

$$\left(\frac{0.003}{2.55}\right) = \left(\frac{\varepsilon_t}{74.37 - 2.55}\right)$$

We obtain $\varepsilon_t = 0.084 > 0.005$. Therefore, assumption of ϕ = 0.90 is correct.

5.6.6.3.6 Step 6: Check for Serviceability

Cracks occur whenever the tension in the gross section exceeds the cracking strength (modulus of rupture) of concrete. One can control or avoid flexural cracking in a concrete component by providing tension reinforcement at certain specified spacing.

The spacing, *s*, of mild steel reinforcement in the layer closest to the tension face:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(AASHTO 5.6.7-1)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)} \qquad \text{(AASHTO 5.6.7-2)}$$

where:

γ_e = exposure factor taken as 0.75 by considering Class 2 exposure condition (CA 5.6.7)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

= 1.5 + 0.69 + 0.69 / 2 = 2.54 in.

h = overall thickness or depth of the component (in.) = 81 in.

f_{ss} = tensile stress in steel reinforcement at the service time limit state not to exceeds $0.6f_y$ (ksi)

Tensile stress in steel reinforcement may be calculated based on the transformed section by the following procedure valid for both rectangular and flanged sections (Figure 5.6.6-14). Moment of inertial of cracked reinforced concrete sections used for calculating stress in reinforcement is discussed in BDP Chapter 5, Section 5.1.4.2.3.

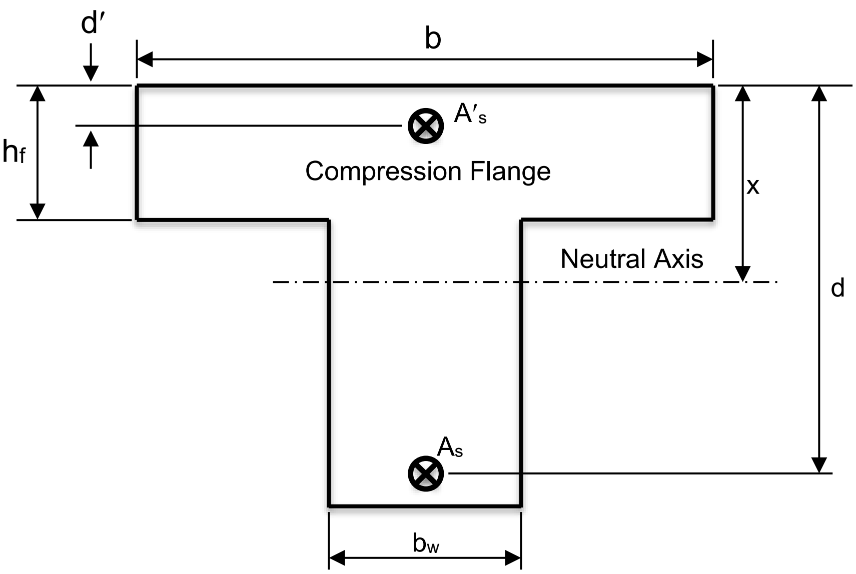


Figure 5.6.6-14 A Typical Flanged Section

Note: Given b, b_w, h_f, d, d', A_s, A'_s, $n = E_s / E_c$, M = applied moment

If $h \neq 0$ and $b \geq \dfrac{2}{h_f^2}\left[n\left(d-h_f\right)A_s-\left(n-1\right)\left(h_f-d'\right)A'_s\right]$, then set $b_w = b$

$$\text{Set} \quad B=\frac{1}{b_w}\left(h_f\left(b-b_w\right)+nA_s+\left(n-1\right)A'_s\right)$$

$$\text{Set} \quad C=\frac{2}{b_w}\left(h_f^2\left(b-b_w\right)/2+ndA_s+\left(n-1\right)d'A'_s\right)$$

$$x=\sqrt{B^2+C}-B \text{ (assumes x} \geq \text{d)}$$

$$I=\frac{1}{3}bx^3-\frac{1}{3}\left(b-b_w\right)\left(x-h_f\right)^3+nA_s\left(d-x\right)^2+\left(n-1\right)A'_s\left(x-d'\right)^2$$

$$f'_c = \frac{M\,x}{I} \text{ = stress in top fiber of compression flange}$$

$$f'_s = \frac{nM\left(x - d'\right)}{I} = nf'_c\left(1 - \frac{d'}{x}\right) \text{ = stress in compression steel}$$

$$f_s = \frac{nM\left(d - x\right)}{I} = nf'_c\left(\frac{d}{x} - 1\right) \text{ = stress in tension steel}$$

- *Check Bottom Reinforcement*

For service load combination, the permit loads are not considered. From Table 5.6.6-2

M_{ser} = 3,377 + 339 + 2,683 = 6,399 kip-ft

Bent cap section:

b_e = 14.8 ft = 177.6 in.

b_w = 96 in.

h_f = 9.0 in.

E_s = 29,000 ksi

E_c = 3,645 ksi

n = E_s / E_c = 7.96

Assume 16 - #5 as crack control reinforcement.

Effective, A_{s1} = (0.31)(16) cos (20°) = 4.66 in.^{2}

Tension reinforcement (bottom), A_{s2} = 1.56 (22) = 34.32 in.^{2}

Compression reinforcement (top), A'_s = 1.56 (14) = 21.84 in.^{2}

where:

d_{e1} = effective depth from extreme comp fiber to the centroid of the crack control reinforcement

= 81 – (1.5 + 0.69 + 0.69/2) = 78.47 in.

d_{e2} = effective depth from extreme comp fiber to the centroid of tension reinforcement (bottom)

= 81 – (5.7 + 1.63) = 73.67 in.

d' = effective depth from extreme comp fiber to the centroid of compression reinforcement (top)

= 5 + 1.63 = 6.63 in.

If $\sum\left(\frac{b_e h_f^2}{2}+(n-1)A_s'(h_f-d')\right) > \sum(nA_s)(d-h_f)$, then neutral axis, y, lies in the flange. Otherwise, the neutral axis lies in the web.

$$\because \ \sum\left(\frac{(177.6)(9)^2}{2} + (7.96-1)(21.84)(9-6.63)\right) = 7{,}553 \ \text{in.}^3$$

$$< (7.96)(4.66)(78.47-9)+(7.96)(34.32)(73.67-9) = 20{,}244 \ \text{in.}^3$$

Therefore, the neutral axis is within web and the compression block has T-section shape.

$$\sum\left((b_e-b_w)h_f\left(y-\frac{h_f}{2}\right)+\frac{b_w y^2}{2} + (n-1)A_s'(y-d')\right) = \sum(nA_s)(d-y)$$

$$(177.6-96)(9.0)(y-\frac{9.0}{2})+96(\frac{y^2}{2})+(7.96-1)(21.84)(y-6.63)$$
$$= 7.96(4.66)(78.47-y)+7.96(34.32)(73.67-y)$$

Solving for y:

y = 14.13 in.

$$I = \frac{b_e y^3}{3} - \frac{(b_e-b_w)(y-h_f)^3}{3} + (n-1)A_s'(y-d')^2 + \sum(nA_s)(d-y)^2$$

$$= \frac{177.6(14.13)^3}{3} - \frac{(177.6-96)(14.13-9.0)^3}{3} + (7.96-1)(21.84)(14.13-6.63)^2$$

$$+\left[7.96(4.66)(78.47-14.13)^2 + 7.96(34.32)(73.67-14.13)^2\right]$$

$$= \ 1293896.18 \ \text{in.}^4 = \ 62.40 \ \text{ft}^4$$

$$f_{ss} = \frac{nM(d-y)}{I} = \frac{7.96(6{,}399)(12)(78.47-14.13)}{62.40(12)^4} = 30.39 \text{ ksi} < 0.6 f_y = 36 \text{ksi}$$

$$\beta_s = 1+\frac{2.54}{0.7(81-2.54)} = 1.046$$

$$s \le \frac{700(0.75)}{1.046(27.0)} - 2(2.54) = 13.51 \text{ in.}$$

- *Check Top Reinforcement*

  Calculations for crack reinforcement at the top of bent cap are not shown. Designer can use a computer program to check serviceability of the section.

  Flexural reinforcements are shown in Figure 5.6.6-15.

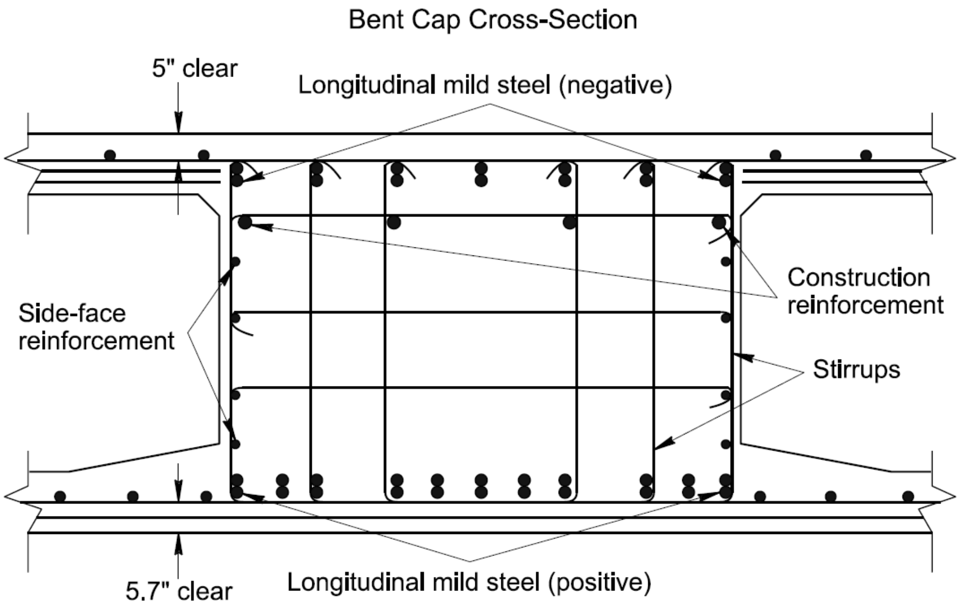


Figure 5.6.6-15 Bent Cap Cross Section

5.6.6.3.7 Step 7: Check for Fatigue

From Section 5.6.6.3.6, I = 62.44 ft^4; y = 14.13 in. $d = d_{e2}$ =73.67 in.

$$f_s = \frac{nM(d-y)}{I}$$

$$f_{s(max)} = \frac{7.96(5{,}097)(12)(73.67-14.13)}{(62.40)(12^4)} = 22.40\,\text{ksi}$$

$$f_{s(\min)} = \frac{7.96(3{,}254)(12)(73.67-14.13)}{(62.40)(12^4)} = 14.30\text{ ksi}$$

$$\gamma(\Delta f) = f_{s(max)} - f_{s(min)} = 22.40 - 14.30 = 8.10\text{ ksi}$$

$$(\Delta F)_{TH} = 24 - 0.33 f_{min} = 24 - 0.33(15.31) = 18.95\,\text{ksi}$$

$$\gamma(\Delta f) = 8.10\text{ ksi} < (\Delta F)_{TH} = 18.95\,\text{ksi} \qquad \text{OK}$$

Fatigue requirement at the midspan is met.

The following critical locations shall be checked for fatigue requirement:

- At maximum difference in positive moment region (bottom steel)
- At maximum difference in negative moment region (top steel)
- Moment reversal (top and bottom steel)

Calculations for fatigue check at top of bent cap are not shown. Designer can use a computer program to check the fatigue limit state of the section.

5.6.6.3.8 Step 8: Perform Shear Design

The shear design equation is as follows:

$$V_u \le V_r = \phi V_n \tag{5.6.6-3}$$

Shear design is performed for critical section located at the face of support using the sectional design model per Article B5.2.

- *Determine* θ *and* β

  Using AASHTO Table B5.2-1 for sections with minimum amount of transverse reinforcement.

  b_v = 8 ft = 96 in.

  d_v, using results from flexural analysis

$$d_v = d_{e2} - \frac{a}{2} = 73.67 - \frac{(0.85)(2.55)}{2} = 72.59 \text{ in.}$$

$$\because \; d_v = 72.59 \text{ in.} > \text{larger} \begin{cases} 0.9d_e = (0.9)(74.37) = 66.93 \text{ in.} \\ 0.72h = (0.72)(81) = 58.3 \text{ in.} \end{cases}$$

  Use d_v = 72.59 in.

  Shear stress:

$$v_u = \frac{V_u}{\phi b_v d_v} = \frac{2{,}076}{0.9(96)(72.59)} = 0.331 \text{ ksi}$$

  Shear stress factor:

$$\frac{v_u}{f'_c} = \frac{0.331}{4} = 0.0828$$

  Determine ε_x at mid-depth

$$\varepsilon_x = \frac{\left(\dfrac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_sA_s + E_pA_{ps}\right)} \qquad \text{(AASHTO B5.2-3)}$$

As there is no prestressing force and axial force in bent cap, the above equation reduces to:

$$\varepsilon_x = \frac{\left(\dfrac{M_u}{d_v} + 0.5V_u\cot\theta\right)}{2\left(E_sA_s\right)} \qquad (5.6.6\text{-}4)$$

For 14-#11 bars,

A_s = area of fully developed steel on flexural tension side of member
= 21.84 in.^{2}

Assuming 0.5cot θ = 1 per Article CB5.2 for the first trial and use absolute values for M_u and V_u for strain calculation.

$$\varepsilon_x = \frac{\left(\dfrac{(1{,}102)(12)}{73.2} + 2{,}076\right)}{2(29{,}000)(21.84)} = 0.001837 = 1.837(10)^{-3}$$

AASHTO Table B5.2-1 lists values of θ and β as function of shear stress factor $\dfrac{v_u}{f_c'}$

and strain at mid-depth of the bent cap ε_x:

AASHTO Table B5.2-1 Values of θ and β for Sections with Transverse Reinforcement (partial)

$\dfrac{v_u}{f_c'}$	$\varepsilon_x \times 1{,}000$								
	≤–0.20	≤–0.10	≤–0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34 2.32	$\boxed{36.7}$ $\boxed{2.18}$
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13

The first trial with the assumption 0.5cot θ = 1, yield ε_x >1, however, the table only provide $\varepsilon_x \leq 1.00$.

Use the $\varepsilon_x \leq 1.00$ column, β = 2.18; θ = 36.7

Since 0.5cot θ = 1 was assumed, re-calculate the ε_x, using θ = 36.7

$$\varepsilon_x = \frac{\left(\dfrac{(1{,}102)(12)}{73.2} + (0.5)(2{,}076)\cot(36.7)\right)}{2(29{,}000)(21.84)} = 0.001281 = 1.281(10)^{-3}$$

Since the re-calculated ε_x do not change β and θ, β = 2.18; θ = 36.7 will be used in shear design.

- *Determine Shear Reinforcement*

Concrete contribution to shear resistance:

$$V_c = 0.0316\beta\sqrt{f_c'}\,b_v d_v = 0.0316(2.18)\sqrt{4}(96)(72.59) = 960 \text{ kips}$$

(AASHTO 5.7.3.3-3)

Shear force on shear stirrups:

$$V_s = \frac{V_u}{\phi} - V_c = \frac{2{,}076}{0.9} - 960 = 1{,}347 \text{ kips}$$

Required shear stirrups:

$$V_s = 1{,}346 = \frac{A_v f_y d_v}{s}\cot\theta = \frac{A_v(60)(72.59)}{s}\cot\left(36.7^o\right)$$

$$\frac{A_v}{s} = 0.230 \text{ in.}^2\text{/in.}$$

Minimum shear reinforcement:

$$\left(\frac{A_v}{s}\right)_{\min} = 0.0316\lambda\frac{\sqrt{f_c'}}{f_y}b_v = 0.0316(1.0)\frac{\sqrt{4}}{60}(96) = 0.1 \text{ in.}^2\text{/in.}$$

(AASHTO 5.7.2.5-1)

Required stirrups spacing:

A_v = use six legs #5

A_v = 0.31 (6) = 1.86 in.^{2}

Required $s = \dfrac{1.86}{0.230} = 8.09$ in.

Use s = 6 in.

- *Check Maximum Spacing (Article 5.7.2.6 and CA 5.7.2.6)*

For $\dfrac{v_u}{f_c'} < 0.125$, $S_{max} = 0.8d_v \leq 18$ in.

For $\frac{v_u}{f'_c} \geq 0.125$, $S_{max} = 0.4d_v \leq 12$ in.

At this particular section:

$$\frac{v_u}{f'_c} = \frac{0.331}{4} = 0.0828, \; S_{max} = 0.8(73.2) = 58.56 \text{ in. but not greater than 18 in.}$$

Maximum spacing allowed = 18 in.

Six legs #5 at 6 in. at the face of column meet this requirement.

- Check Tenth Points along Bent Cap

For V_u, M_u, θ, β, V_c, V_s, S, A_{vmin}, and S_{max}, Figure 5.6.6-16 shows stirrup spacing along the bent cap length.

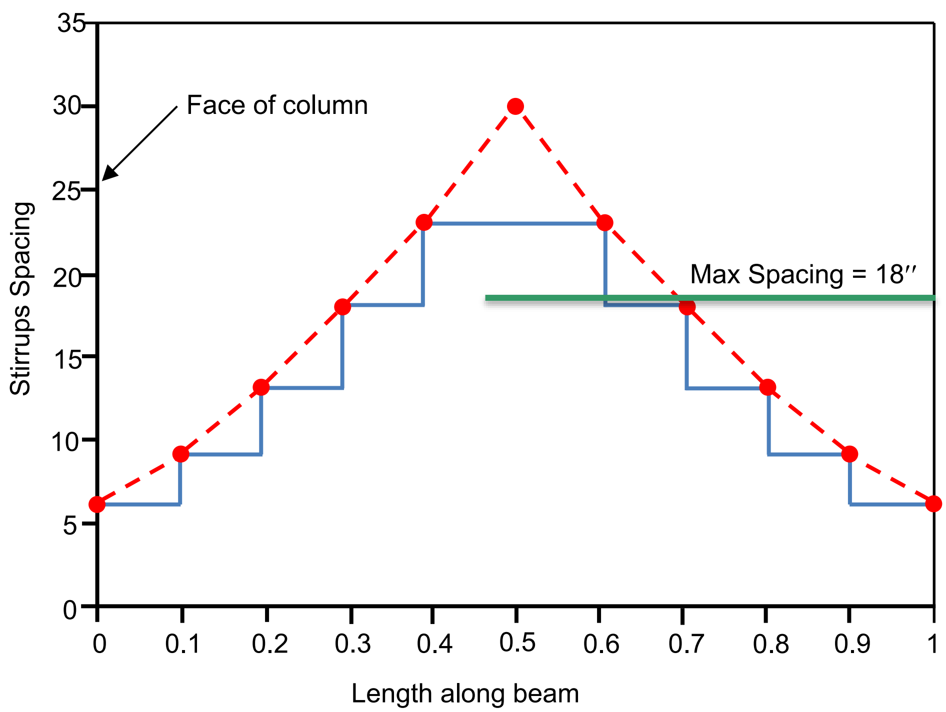


Figure 5.6.6-16 Stirrup Spacing along Bent Cap Length

- *Check Longitudinal Reinforcement (Shear-Flexural Interaction) at the Face of Support*

For a bent cap without prestressing steel and axial force, the longitudinal steel shall satisfy:

$$A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + \left(\left| \frac{V_u}{\phi_v} \right| - 0.5V_s \right) \cot\theta \qquad \text{(AASHTO 5.7.3.5-1)}$$

Note: In practice, after the design is complete, engineer must check seismic design requirements as per SDC and check the longitudinal steel in the bent cap to handle seismic moments.

Check case of the maximum shear and the associated moment:

$V_{u(max)}$= -2,007 kip

$M_{u(assoc)}$ = 1,835 kip-ft

Compute ε_x and θ for this particular location under this loading:

For 22- #11, $A_{s(bot)}$ = 34.32 in.^{2}

d_v = 73.67 – (0.85)(4.33)/2 = 71.83 in.

$$\frac{v_u}{f'_c} = 0.0876\,;$$

Using Eq. 5.6.6-4 , we obtain ε_x = 0.00116;
For AASHTO Table B5.2-1, θ = 36.7°

$$V_s = \frac{A_v f_y d_v}{s}\cot\theta = \frac{1.86(60)(71.83)}{6}\cot\left(36.7^o\right)$$

$$= 1{,}792.4 \text{ kips} \quad < \frac{V_u}{\phi} = \frac{2{,}007}{0.9} = 2{,}230 \text{ kips}$$

$$A_s f_y \geq \frac{(1{,}835)(12)}{(0.9)(71.83)} + \left(\frac{2{,}007}{0.9} - (0.5)(1{,}792.4)\right)\cot\left(36.7^o\right)$$

$$A_s(60) \geq 340.62 + 1{,}613.07 = 1{,}953.69 \text{ kips}$$

$$A_{s(req)} = 32.56 \text{ in.}^2 < A_{s(bot)} = 34.32 \text{ in.}^2$$

The above procedure can be used to check for cases of the maximum negative moment and the associated shear

$M_{u(max)}$ = -4,813 kip-ft

$V_{u(assoc)}$= -1,405 kip

and the maximum positive moment and the associated shear

$M_{u(max)}$ = 1,835 kip-ft

$V_{u(assoc)}$= -2,007 kip

Moment and shear in this case is similar to maximum shear and associated moment case.

Note: Using the same process, one can determine the size and spacing of the stirrups at other locations.

5.6.6.3.9 Step 9 - Determine Additional Details

Additional details are discussed as follows.

- *Construction Reinforcement*

  As shown in Figure 5.6.6-17, the concrete in the superstructure is poured in two stages during the construction process. It is a Caltrans practice to assume that 10 ft of the soffit on each side of the bent cap contributes to this dead load:

  Stage 1 (first pour) includes the soffit slab and the girder stems. The deck slab is not included.

  Stage 2 (second pour) includes the deck slab.

Height of the Stage 1 pour = (6.75)(12) – 9 – 3 = 69 in. = 5.75 ft

Width of the bent cap = 8 ft

Dead load due to exterior girder stem

$$DL = \left[\frac{\left(5.75 - \frac{8.25}{12}\right)}{\cos 26.6^\circ (1)} \right](10 + 10)(0.15) = 19 \text{ kip}$$

Dead load due to cap and soffit

Width of soffit slab causing negative moment = 10 + 10 = 20 ft

$$DL = \left\{(5.75)(8) + 20\left(\frac{8.25}{12}\right)\right\}(0.15) = 9 \text{ kip per ft}$$

Column diameter = 6.0 ft

Negative moment at the face of the column

$$M_u = 1.25\left\{\left[19\left(7.12 - \frac{6.0}{2}\right)\right] + \left[9\frac{\left(7.12 - \frac{6.0}{2}\right)^2}{2}\right]\right\}$$

$$= 193 \text{ kip-ft}$$

Note: Assume a reduced value for f'_c as the concrete has not reached its specified compressive strength at the Stage 1 pour. Hence, use f'_c = 2.5 ksi. Caltrans standard practice is to provide four #10 bars as minimum construction reinforcement in the bent cap.

Since there is no prestressing steel or compression reinforcement, for simplicity of calculations, the overhangs will be neglected, and nominal moment is calculated by:

$$M_n = A_s f_s \left(d_s - \frac{a}{2} \right) + 0.85 f'_c \left(b - b_w \right) h_f \left(\frac{a}{2} - \frac{h_f}{2} \right) \tag{5.6.6-11}$$

b = 96 in.

d_s = 69 – 3 = 66 in.

A_s = 1.27(4) = 5.08 in.^{2}

β_1 = 0.85

c = (5.08)(60) / [(0.85)(2.5)(0.85)(96)] = 1.76 in.

a = $c\ \beta_1$ = 1.494 in.

M_n = 5.08 (60)(66-1.494/2) = 19,889 kip-in. = 1,657 kip-ft

$M_r = \phi M_n = (0.9)(1{,}657) = 1{,}491$ kip-ft $> M_u =$ 193 kip-ft OK

Provide 4 # 10 (A_s = 5.08 in.^{2}) as construction reinforcement.

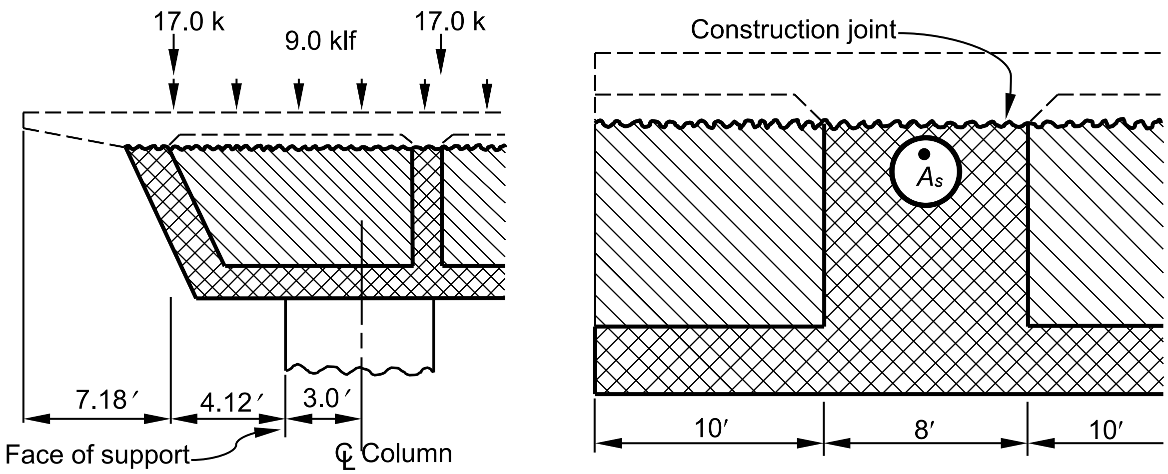


Figure 5.6.6-17 Concrete Pour Stages

- *Side Face / Skin Reinforcement*

Per Caltrans SDC 7.4.5.2 the side face reinforcement is 10 percent of the maximum longitudinal reinforcement.

Maximum longitudinal reinforcement, $A_{s(bot)}$ = 34.32 in.^{2}

Side face reinforcement: $A_s^{sf} = 0.1(34.32) = 34.32$ in.^{2}

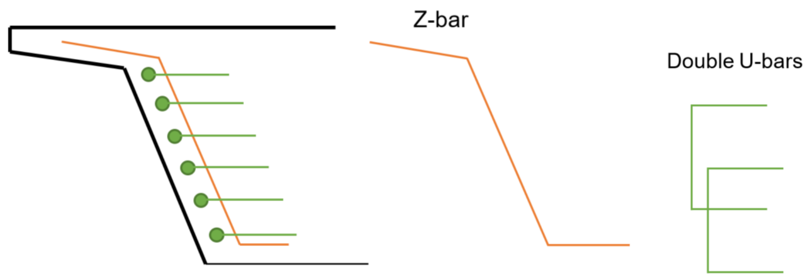
Provide 10 - #6 bars. Since the construction reinforcement is also provided, 2 -

#10 bars would also count as side face reinforcement. So, provide 4 - #6 bars on each side of the bent cap.

Spacing = {(5.75)(12) – (5.7+ 1.63) – 3} / 5 = 11.73 in. < 12 in.   OK

- *End Reinforcement*

End reinforcements as shown in Figure 5.6.5-18 should be provided as a crack control measure, as discussed in Section 5.6.4.3.



U-bars in the Bent Cap

Figure 5.6.6-18 End Reinforcements

The U-bars are designed for shear friction. Since the ben cap width is larger than 7 ft., the double U-bar is used in this example per past practice. Length of U-bars should extend a development length beyond the inside face of the exterior girder.

5.6.7 DROP BENT CAP DESIGN EXAMPLE

5.6.7.1 Bent Cap Data

A three-span bridge with reinforced concrete drop bent caps is shown in Figures 5.6.7-1 through 5.6.7-4.

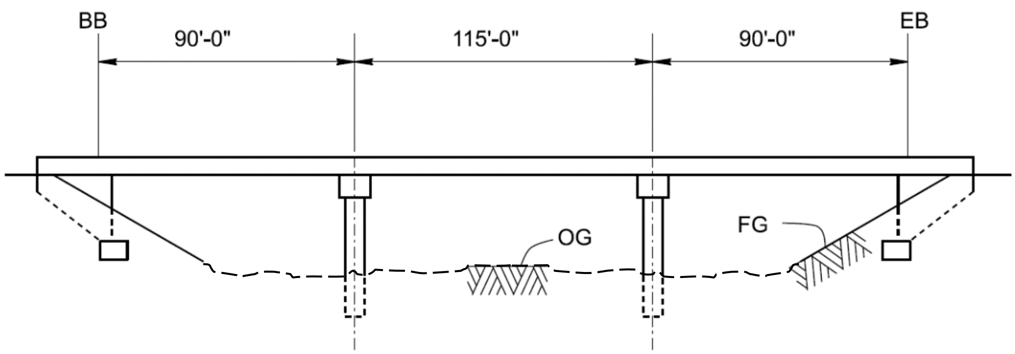


Figure 5.6.7-1 Elevation

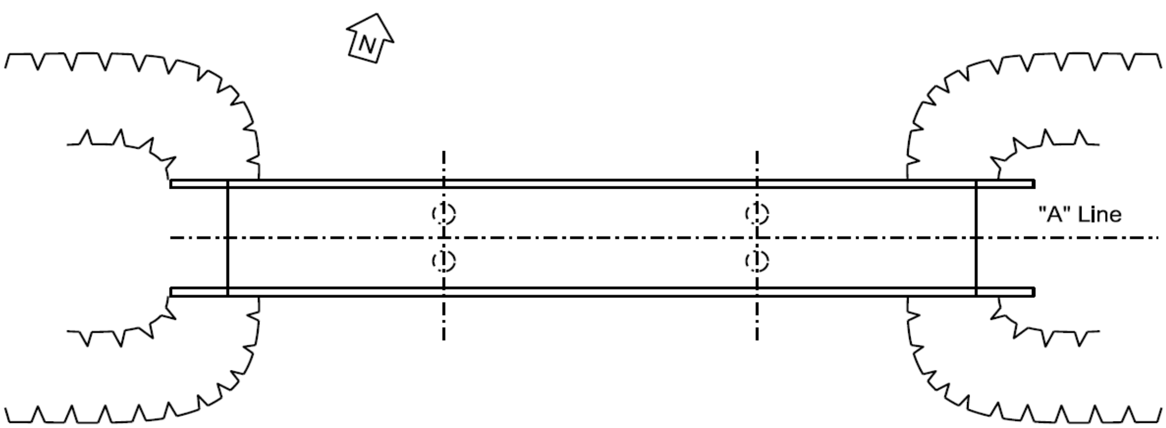


Figure 5.6.7-2 Plan

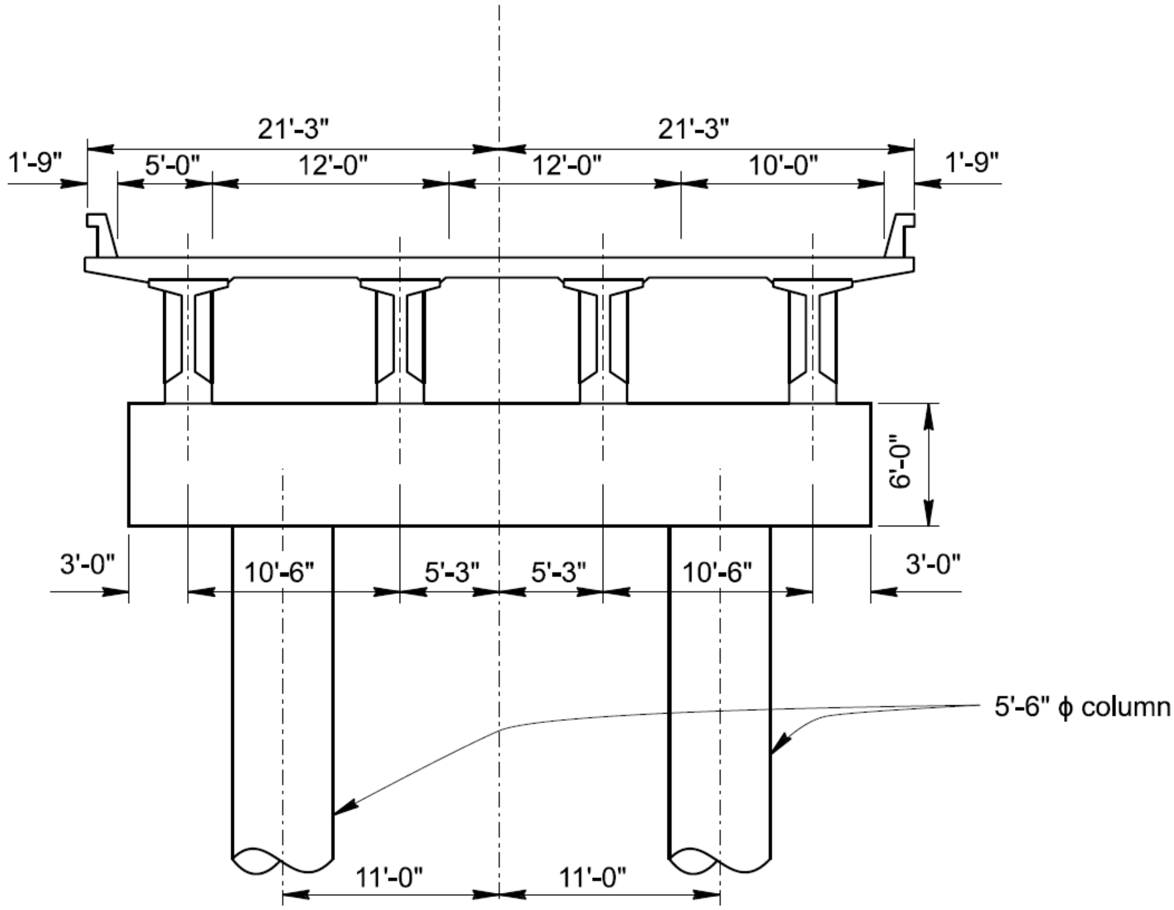


Figure 5.6.7-3 Typical Section

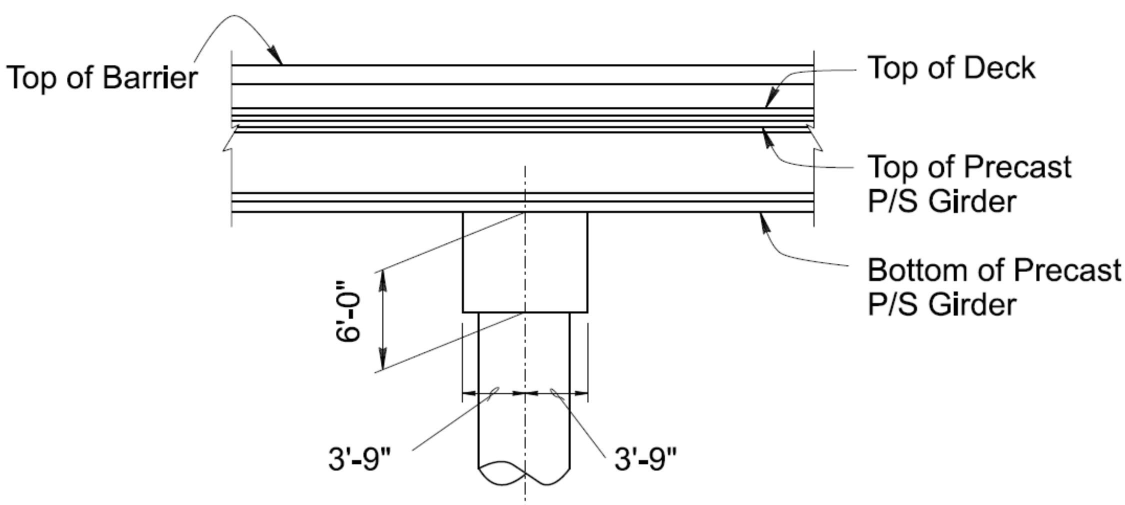


Figure 5.6.7-4 Side View

5.6.7.2 Design Requirements

Perform the structural analysis, flexural, and shear design for the bent cap as shown in Figures 5.6.7-1 to 5.6.7-4 in accordance with the AASHTO-CA BDS-8.

5.6.7.3 Design

5.6.7.3.1 Step 1: Determine Geometry of Bent

Member Proportioning

The bent cap depth should be deep enough to develop the column longitudinal reinforcement without hooks (SDC 7.3.3 and SDC 8.3.1). The minimum bent cap width required for adequate joint shear transfer shall be column sectional width in the direction of interest, plus two feet (SDC 7.4.3).

$$d_{cap} = 6 \text{ ft}$$

$$B_{cap} = 5.5 + 2 = 7.5 \text{ ft}$$

Classification of Bent Cap (AASHTO 5.7.1.1)

Bent caps may fall under the classification of flexural beam or deep beam. The classification dictates the type of analytical theory that would most accurately estimate the internal forces of the bent cap.

Bent caps have typically been designed by using the sectional method which has been proved to be acceptable as historical data does not suggest design inadequacies in Caltrans. This design example will assume all the requirements in CA 5.8.4.1 are met and follow the sectional method for its conservativeness and ease of application. Caltrans will continue to use the sectional method until the strut-and-tie method is adopted agency-wide.

5.6.7.3.2 Step 2: Material Properties

Material properties are as follows:

$$f'_c = 4 \text{ ksi}$$

$$f_y = 60 \text{ ksi}$$

$$E_s = 29{,}000 \text{ ksi}$$

5.6.7.3.3 Step 3: Determine Bent reactions due to Permanent and Live Loads

CTBridge, LEAP Bridge, or other computer analysis programs can be used to determine bent reactions due to the dead and live loads along the length of the bridge. For the bent cap design, the analysis is performed with only a single lane and/or truck so that wheel line loads may be generated and subsequently implemented in the frame analysis of the bent cap.

Generally, dead loads such as $DC_{superstructure}$ and DW are distributed by tributary area (or width) for PC/PS I girder, CA wide flange, bulb T girder, and steel girder bridges. DC_{bent_cap}, however, is distributed along the length of the bent cap as a tributary load. In very stiff superstructures, such as cast-in-place prestressed concrete box girders, $DC_{superstructure}$ and DW may be distributed equally despite varying girder spacing. For this drop cap example, the deck, girders, bent cap, and columns will be modeled as individual elements.

Based on the longitudinal analysis, the following dead loads are applied to the bent cap analytical model as shown in Figure 5.6.7-5.

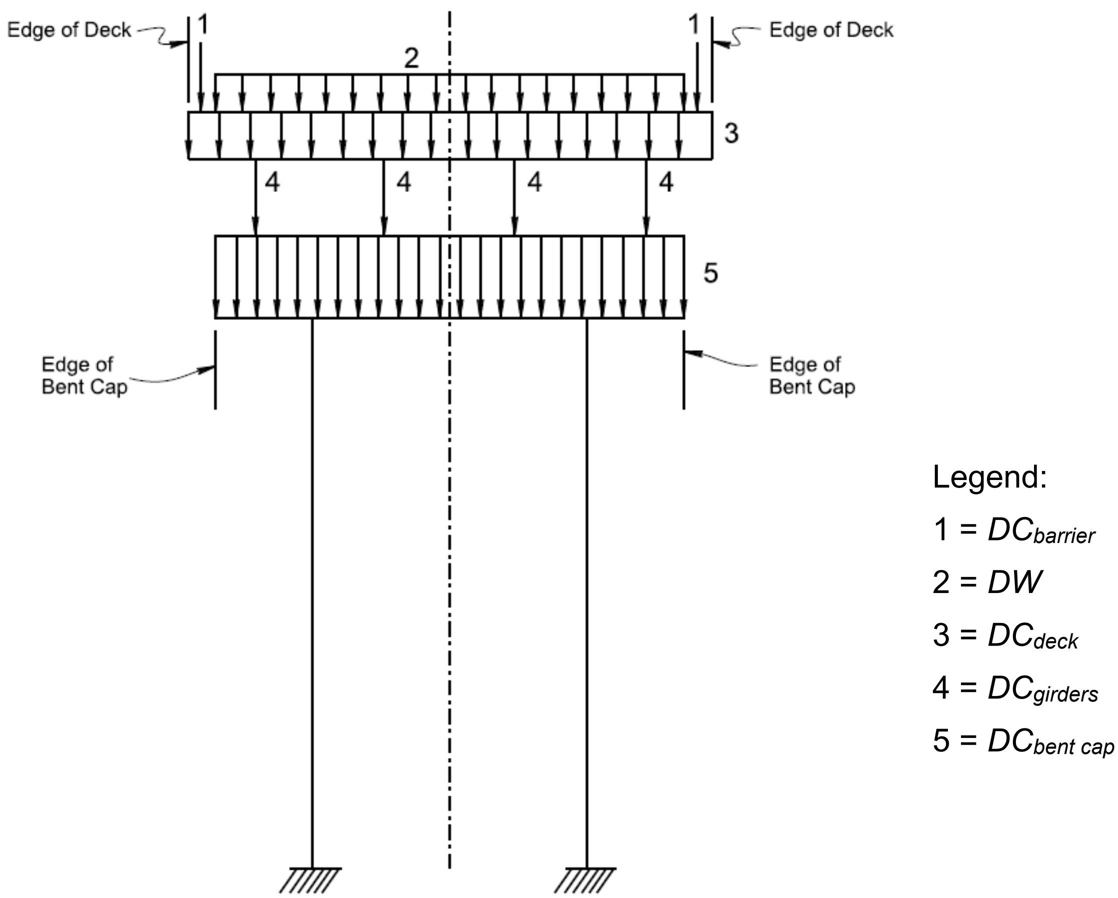


Figure 5.6.7-5 Dead Loads in Elevation View

$P_{DC\_barrier}$ = 72.6 kip

w_{DC_deck} = 12.8 kip/ft

w_{DW} = 4.4 kip/ft

P_{DC_girder} = 178 kip

$w_{DC\_bent\_cap}$ = 7.2 kip/ft

Force effects, on the bent cap, from live loads are determined similarly to the methods used for the longitudinal analysis. The live loads are discretized into wheel line loads with fixed and variable spacing to represent spacing between wheel lines, as well as spacing between trucks and lanes. Influence lines are generated to determine the governing force effects on the bent cap element. We will begin this process by generating wheel line loads from CTBridge program results.

Unfactored Bent Reactions from Longitudinal Analysis

For each design vehicular live load, the unfactored bent reactions at the bent can be obtained from the output of the longitudinal analysis. Results shown in Figures 5.6.7-6 to 5.6.5-8 are due to a single truck or lane load. The location designating "Col Bots" and "Col Tops" are the force effects to the bottoms and tops of all columns from the single truck or lane load. Results from three live loads, LRFD design vehicle (also known as HL-93), LRFD permit vehicle (also known as P-15), and the LRFD fatigue vehicle are shown in Figures 5.6.7-6 to 5.6.7-8, respectively. The LRFD design vehicle consists of a truck and a lane. We are only interested in the maximum axial load from the truck or lane, and those values are boxed accordingly and shown as follows:

$LL_{HL\text{-}93\_truck}$ = design truck (1 lane only) (AASHTO 3.6.1.2.2)

$LL_{HL\text{-}93\_lane}$ = lane load (1 lane only) (AASHTO 3.6.1.2.4)

LL_{permit} = permit vehicle (1 lane only) (CA 3.6.1.8)

$LL_{fatigue}$ = fatigue vehicle (1 lane only) (AASHTO and CA 3.6.1.4)

Live Load - Controlling Unfactored Bent Reactions

Bent 2 Reactions - LRFD Design Vehicle
No Dynamic Load Allowance - Single Lane

Location	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip·ft	MZ kip·ft
Col Bots	AX-	Truck	-117.24	0.03	-0.10	2.47	1.96
		Lane	-115.19	0.05	-0.24	5.65	3.73
Col Bots	AX+	Truck	13.43	0.02	0.23	-4.97	1.27
		Lane	10.58	0.01	0.18	-3.93	1.03
Col Bots	MY-	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-47.43	-0.00	0.46	-9.70	-0.13
Col Bots	MY+	Truck	-67.31	0.07	-0.62	13.71	5.56
		Lane	-69.97	0.07	-0.54	12.05	5.31
Col Bots	MZ-	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-54.02	-0.02	0.30	-6.24	-1.26
Col Bots	MZ+	Truck	-63.92	0.07	-0.61	13.69	5.57
		Lane	-63.39	0.08	-0.38	8.59	6.43
Col Bots	VY-	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-54.02	-0.02	0.30	-6.24	-1.26
Col Bots	VY+	Truck	-63.92	0.07	-0.61	13.69	5.57
		Lane	-63.39	0.08	-0.38	8.59	6.43
Col Bots	VZ-	Truck	-67.31	0.07	-0.62	13.71	5.56
		Lane	-69.86	0.07	-0.54	12.05	5.31
Col Bots	VZ+	Truck	-74.82	-0.02	0.43	-9.02	-1.52
		Lane	-47.55	-0.00	0.46	-9.70	-0.13
Col Tops	AX-	Truck	-117.24	0.03	-0.10	-5.00	0.10
		Lane	-115.19	0.05	-0.24	-11.58	0.18
Col Tops	AX+	Truck	13.43	0.02	0.23	11.90	0.06
		Lane	10.58	0.01	0.18	9.39	0.05
Col Tops	MY-	Truck	-67.31	0.07	-0.62	-31.39	0.27
		Lane	-69.90	0.07	-0.54	-27.28	0.25
Col Tops	MY+	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-47.50	0.00	0.46	23.80	0.00
Col Tops	MZ-	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-54.02	-0.02	0.30	15.41	-0.06
Col Tops	MZ+	Truck	-63.92	0.07	-0.61	-31.24	0.27
		Lane	-63.39	0.08	-0.38	-18.88	0.31
Col Tops	VY-	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-54.02	-0.02	0.30	15.41	-0.06
Col Tops	VY+	Truck	-63.92	0.07	-0.61	-31.24	0.27
		Lane	-63.39	0.08	-0.38	-18.88	0.31
Col Tops	VZ-	Truck	-67.31	0.07	-0.62	-31.39	0.27
		Lane	-69.86	0.07	-0.54	-27.27	0.26
Col Tops	VZ+	Truck	-74.82	-0.02	0.43	22.66	-0.07
		Lane	-47.55	-0.00	0.46	23.80	-0.01

Figure 5.6.7-6 LRFD Design Vehicle

Bent 2 Reactions - LRFD Permit Vehicle
No Dynamic Load Allowance - Single Lane

Location	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip·ft	MZ kip·ft
Col Bots	AX-	Truck	-368.22	0.06	-0.19	4.81	4.64
Col Bots	AX+	Truck	43.63	0.05	0.75	-16.19	4.16
Col Bots	MY-	Truck	-250.44	-0.06	1.28	-26.59	-4.38
Col Bots	MY+	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	MZ-	Truck	-250.44	-0.06	1.28	-26.59	-4.38
Col Bots	MZ+	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	VY-	Truck	-250.44	-0.06	1.28	-26.59	-4.38
Col Bots	VY+	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	VZ-	Truck	-250.93	0.23	-2.05	45.64	17.94
Col Bots	VZ+	Truck	-264.36	-0.06	1.28	-26.59	-4.35
Col Tops	AX-	Truck	-368.22	0.06	-0.19	-9.27	0.23
Col Tops	AX+	Truck	43.63	0.05	0.75	38.71	0.20
Col Tops	MY-	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	MY+	Truck	-264.36	-0.06	1.28	66.83	-0.21
Col Tops	MZ-	Truck	-250.44	-0.06	1.28	66.81	-0.21
Col Tops	MZ+	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	VY-	Truck	-250.44	-0.06	1.28	66.81	-0.21
Col Tops	VY+	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	VZ-	Truck	-250.93	0.23	-2.05	-104.61	0.87
Col Tops	VZ+	Truck	-264.36	-0.06	1.28	66.83	-0.21

Figure 5.6.7-7 LRFD Permit Vehicle

Bent 2 Reactions - LRFD Fatigue Vehicle
No Dynamic Load Allowance - Single Lane

Location	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip·ft	MZ kip·ft
Col Bots	AX-	Truck	-70.63	0.01	-0.03	0.66	0.50
Col Bots	AX+	Truck	9.59	0.01	0.16	-3.56	0.87
Col Bots	MY-	Truck	-52.71	-0.01	0.31	-6.42	-1.08
Col Bots	MY+	Truck	-47.45	0.05	-0.44	9.80	3.97
Col Bots	MZ-	Truck	-52.47	-0.01	0.31	-6.42	-1.08
Col Bots	MZ+	Truck	-46.71	0.05	-0.44	9.80	3.98
Col Bots	VY-	Truck	-52.47	-0.01	0.31	-6.42	-1.08
Col Bots	VY+	Truck	-46.71	0.05	-0.44	9.80	3.98
Col Bots	VZ-	Truck	-48.58	0.05	-0.44	9.80	3.97
Col Bots	VZ+	Truck	-53.09	-0.01	0.31	-6.42	-1.08
Col Tops	AX-	Truck	$\boxed{-70.63}$	0.01	-0.03	-1.42	0.02
Col Tops	AX+	Truck	9.59	0.01	0.16	8.52	0.04
Col Tops	MY-	Truck	-48.63	0.05	-0.44	-22.44	0.19
Col Tops	MY+	Truck	-53.09	-0.01	0.31	16.13	-0.05
Col Tops	MZ-	Truck	-52.47	-0.01	0.31	16.11	-0.05
Col Tops	MZ+	Truck	-46.71	0.05	-0.44	-22.38	0.19
Col Tops	VY-	Truck	-52.47	-0.01	0.31	16.11	-0.05
Col Tops	VY+	Truck	-46.71	0.05	-0.44	-22.38	0.19
Col Tops	VZ-	Truck	-48.58	0.05	-0.44	-22.44	0.19
Col Tops	VZ+	Truck	-53.09	-0.01	0.31	16.13	-0.05

Figure 5.6.7-8 LRFD Fatigue Vehicle

The lane load is considered uniformly distributed over a 10-ft width. However, we will simplify the analysis by combining the HL-93 lane load with the HL-93 truck wheel line loads. Per AASHTO 3.6.1.2.4, the *IM* shall be applied only to the truck load. The dynamic load allowance is applied during the combination of the truck and lane load are:

$IM_{HL\text{-}93\_truck} = 1.33$; $IM_{Permit} = 1.25$ (CA Table 3.6.2.1-1)

$LL_{HL\text{-}93\_single\_truck} = 117.24$ kip

$LL_{HL\text{-}93\_single\_lane} = 115.19$ kip

$LL_{permit\_single\_truck} = 368.22$ kip

$LL_{fatigue\_single\_truck} = 70.63$ kip

$$LL_{HL93\_wheel\_line} = \frac{LL_{HL93\_single\_truck}\left(IM_{HL93\_truck}\right) + LL_{HL93\_single\_lane}}{2}$$

$$= \frac{117.24(1.33) + 115.19}{2} = 135.6 \text{ kip}$$

$$LL_{Permit\_wheel\_line} = \frac{LL_{Permit\_single\_truck}\left(IM_{permit}\right)}{2} = \frac{368.22(1.25)}{2} = 230.1 \text{ kip}$$

$$LL_{Fatigue\_wheel\_line} = \frac{LL_{Fatigue\_single\_truck}}{2} = \frac{70.63}{2} = 35.3 \text{ kip}$$

5.6.7.3.4 Step 4: Perform Transverse Analysis

Determine Number of Live Load Lanes

Maximum number of whole live load lanes is obtained as:

Clear bridge width between curbs and/or barriers = 39 ft.

N = Integer part of (w/12) = Integer part of (39/12) = 3

Moving Live Load Transverse Analysis for Traffic Lanes

For this drop cap example, the design vehicle variations are shown below. This is based on the maximum number of live load lanes—a total of three—that can possibly fit in the clear roadway width. Note that the multiple presence factor, m, for one lane of permit vehicle is 1.0 (CA Article 3.6.1.8.2), and the factor does not apply to the fatigue vehicle (Article 3.6.1.1.2). Figure 5.6.7-9 shows traffic lanes with a multiple presence factor.

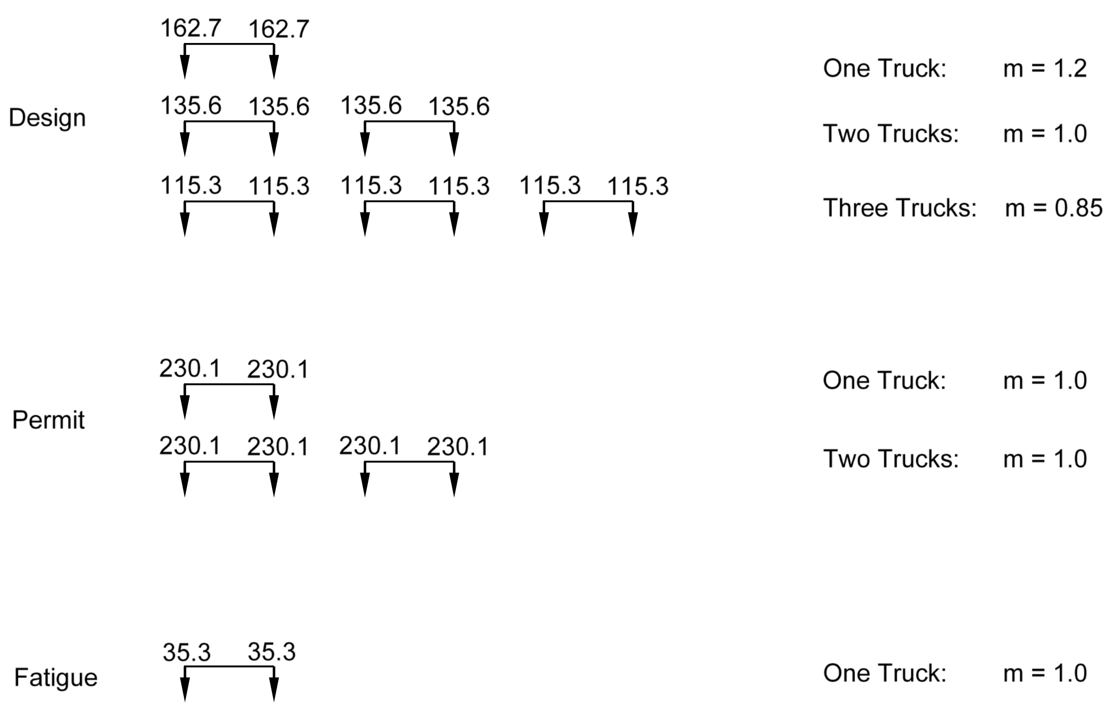


Figure 5.6.7-9 Types and Number of Live Loads to Apply to Bent Cap

Both the design truck and the 10 ft loaded width in each lane are positioned along the clear bridge width to produce the maximum force effects. The design load is positioned transversely such that the center of any wheel load is not closer than 2 ft from the edge of the design lane (Article 3.6.1.3.1).

For the moving load transverse analysis, the wheel lines may move anywhere within the

12 ft lane as long as Article 3.6.1.3.1 is satisfied. Figures 5.6.7-10 and 5.6.7-11 show possible wheel line placement within the same 12-ft lane configuration. The designer must determine the placement of wheel lines that produces the maximum force effects in the bent cap.

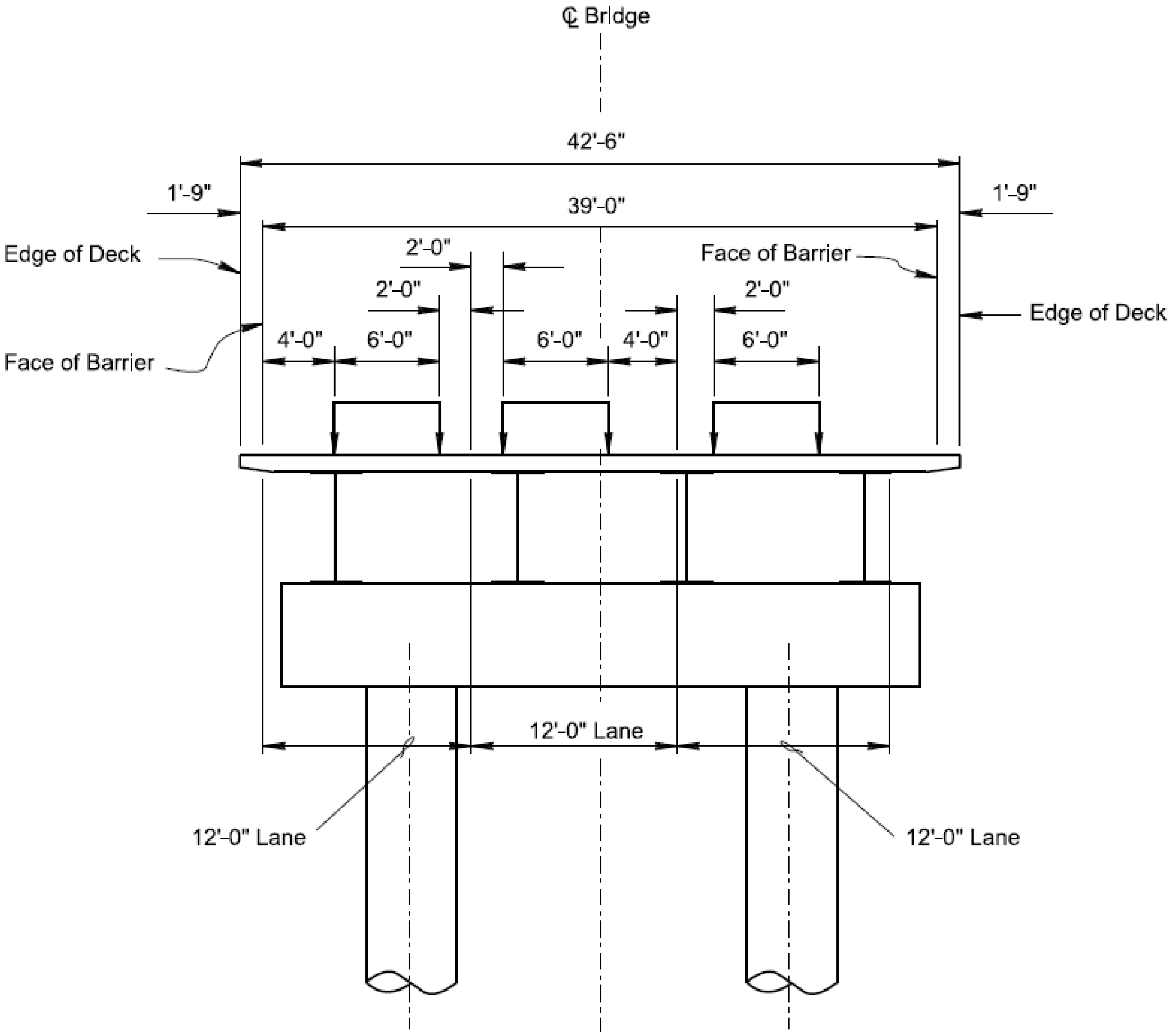


Figure 5.6.7-10 Example Placement of Live Load

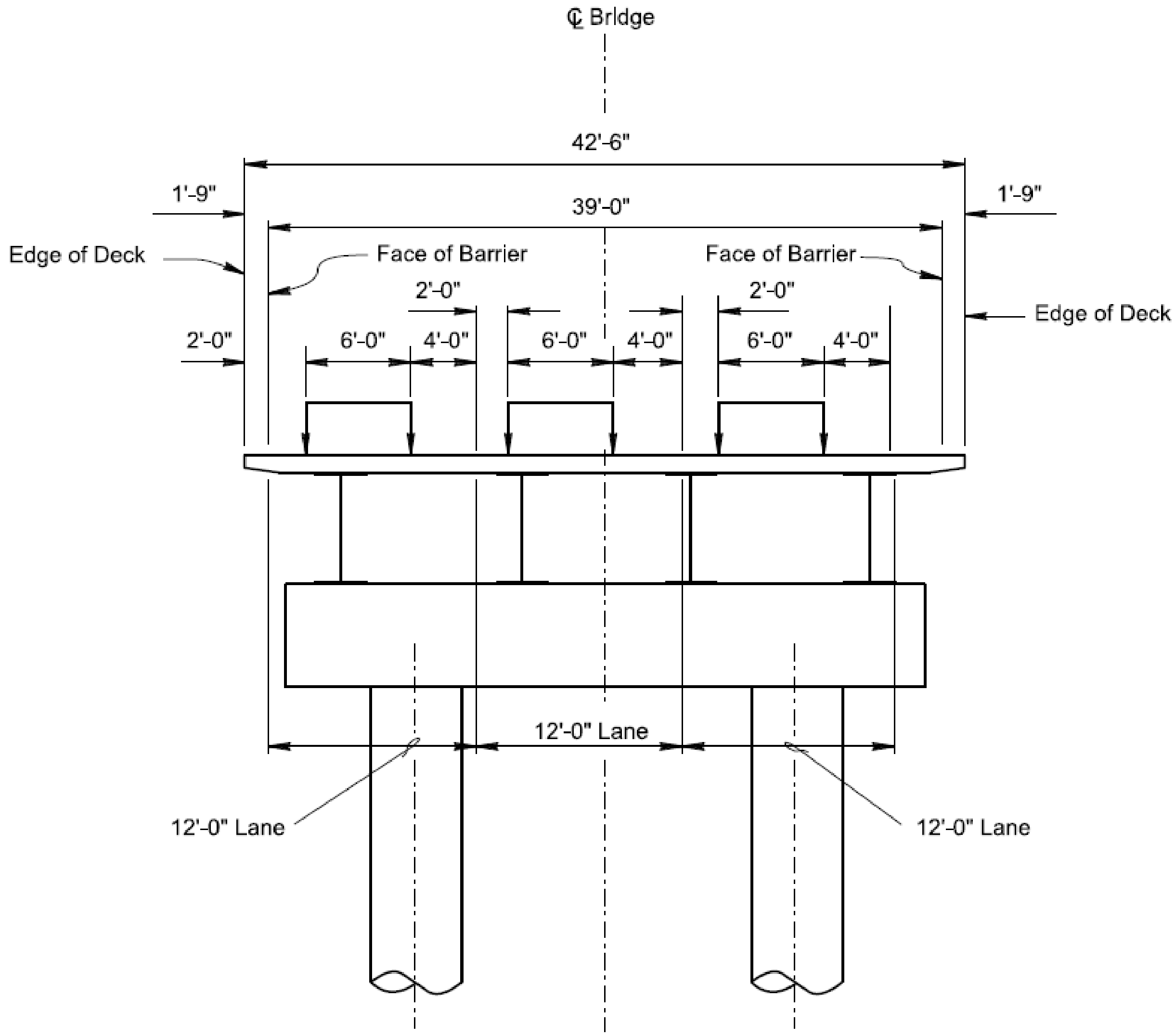


Figure 5.6.7-11 Another Example of Placement of Live Load

Additionally, the 12-ft lanes may move within the confines of the clear bridge width as long as no 12-ft lane overlaps another 12-ft lane. The designer must place the 12-ft lane, as well as the wheel lines, to garner the maximum force effects on the bent cap.

The designer should consider analyzing the transverse model with one, two, and three truck configurations since it is not entirely evident that three trucks will always result in the maximum force effects in the bent cap. For the cap overhang, a single vehicle placed as close to the edge of the design lane as possible may result in the maximum negative moment demand. Note that by placing only one vehicle, the multiple presence factor, *m*, is maximized and will result in a higher negative moment demand than placing two trucks with a lower multiple presence factor. For a bent cap supported by multiple columns, it is advisable to use a structural analysis program, such as CSiBridge, capable of generating combinations of lane configurations and influence lines from moving live loads. CSiBridge is used for this example.

The geometry and model as shown in Figure 5.6.7-12 consider the deck and girders atop the bent cap. Some designers may choose to construct a hybrid frame in which the deck, girders, and bent cap are represented by an integrated horizontal frame member. The point of the fixity of the shaft from the finished grade is assumed to be 16 ft, approximately 3 times of the shaft diameter. For the purpose of maintaining a simplified representation, we are opting to keep the members separate.

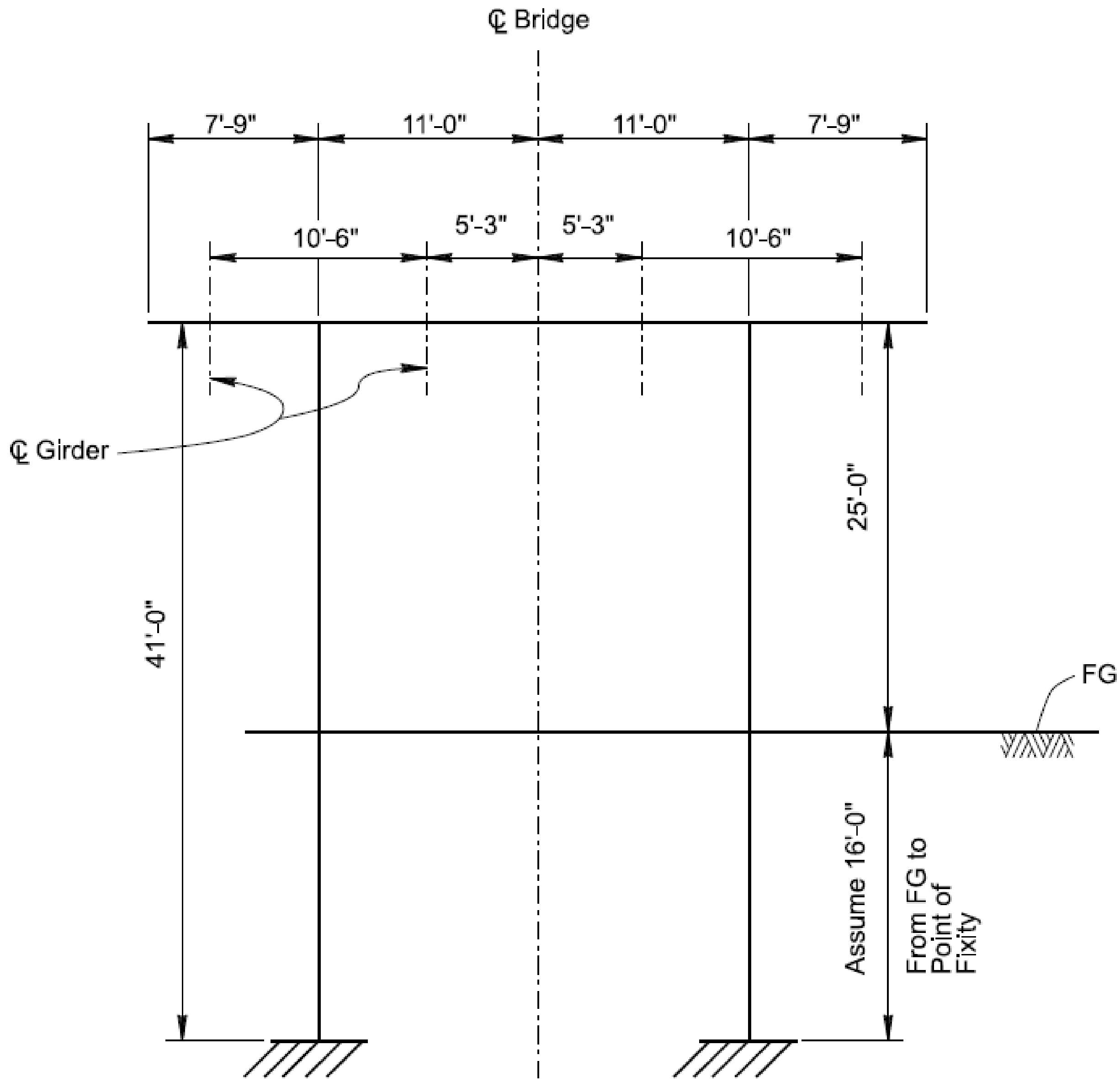


Figure 5.6.7-12 Geometric Model of Bent Cap

For each vehicular load (HL-93, Permit, and Fatigue), the output will be organized in the following manner:

Max *M3*, associated *V2*

Min *M3*, associated *V2*

Max *V2*, associated *M3*

Min *V2*, associated *M3*

Breaking down the force effects in such a manner allows us to combine them with dead loads and apply the appropriate load factors. The associated force effects are necessary for checking moment-shear interaction (Article 5.7.3.5).

The bent cap locations of interest will be positive moments at the midspan, negative moments at faces of the column, and shear at faces of the column.

Maximum positive moments at the midspan obtained from CSiBridge output are summarized in Tables 5.6.7-1.

Table 5.6.7-1 Maximum Positive Moments at Midspan

Dead Loads	Live Load (*M*3)	Live Load (Associated *V*2)
M_{DC_33} = 71.2 kip-ft V_{DC_33} = 4.2 kip M_{DW_33} = 73.8 kip-ft V_{DW_33} = 0 kip	$M_{HL\text{-}93\_33}$ = 1181.9 kip-ft M_{Permit_33} = 2005.6 kip-ft	$V_{HL\text{-}93\_33}$ = 2 kip V_{Permit_33} = 3.4 kip

This example calculation will evaluate the bent cap at the right face of the left column. Maximum negative moments for the right face of the left column obtained from CSiBridge are summarized in Table 5.6.7-2.

Table 5.6.7-2 Maximum Negative Moment at the Right Face of the Left Column

Dead Loads	Live Load (*M*3)
M_{DC_34} = -1127.5 kip-ft V_{DC_34} = 344.5 kip M_{DW_34} = -82.6 kip-ft V_{DW_34} = 47.3 kip	$M_{HL\text{-}93\_34}$ = -904.9 kip-ft M_{Permit_34} = -1530.7 kip-ft $M_{Fatigue\_34}$ = -196.3 kip-ft

5.6.7.3.5 Step 5: Perform Flexure Design

When designing for flexure, the designer should consider loads discussed thoroughly in AASHTO Articles 3.6, 3.7, 3.8, 3.9, 3.10, 3.11, 3.12, 3.13, and 3.14. Of those loads, *CR*, *SH*, wind load on live load (*WL*), wind load on structure (*WS*), temperature loads (*TU*), and differential settlement (*SE*) can impose force effects in the bent cap.

By virtue of experience, flexural and shear design is almost always governed by the Strength I and Strength II limit states. Since *WL* and *WS* are not considered in either of these load combinations (CA Table 3.4.1-1), force effects from wind are not computed. However, uniform temperature is considered in both Strength I and Strength II limit states, but because of the relatively short distance between the two columns, it is anticipated that the force effects generated by *TU* is insignificant.

Differential shrinkage pertains to strains generated between material of different age or composition. This example drop cap will be built monolithically; therefore, *SH* is not considered. Creep is a force effect that is generated by prestressed concrete elements. This example drop cap will be conventionally reinforced with mild reinforcement; therefore, *CR* is also not considered.

For this example, differential settlement will not be considered. Generally, the geotechnical engineer dictates the consideration of differential settlement. *SE* may impose force effects in the bent cap if the soil profiles between the two columns differ, thereby causing one column to settle more than the other column.

The detailed calculations for the flexure design are shown in Step 5 of the previous example, and therefore are not repeated here.

Regardless of which computer program is used to perform the flexure design, the bar size and spacing configurations should satisfy the strength, service, fatigue and extreme limit states as well as crack control (Article 5.6.7) and minimum reinforcement (Article 5.6.3.3) requirements.

The following load combinations are checked for this example. Note: some of the abbreviations are just for this bridge design practice chapter.

S-As	=	Service I, bar spacing for crack control	(AASHTO 5.6.7)
F-As	=	Fatigue I, fatigue stress in mild steel	(AASHTO 5.5.3.2)
Str-I	=	HL-93 loads	
Str-II	=	Permit loads	
Str-III	=	No HL-93 loads, wind > 55 mph	
Str-IV	=	No HL-93 loads, governs when DL to LL ratio is high	
Str-V	=	HL-93 loads, wind = 55 mph	
Ext-I	=	Earthquake	
Ext-II	=	Ice, vehicle, or vessel collision	
Arb-I	=	User defined load	
Min1	=	Minimum reinforcement requirement, M_{cr}	
Min2	=	Waiver of minimum reinforcement requirement, 1.33 M_u	

For flexural design in positive bending, analysis shows that Min2 governed, and bar sizes #7, #8, #9, #10, and #11 provide acceptable capacity while satisfying bar spacing requirements. A total of 16 #8 bars are used.

For flexural design in negative bending, Min2 also governs, and bar sizes #8, #9, #10, #11, and #14 provide acceptable capacity while satisfying bar spacing requirements. A 16 - #10 bars is selected, so the top bars and bottom bars align for stirrup fit. It is a good practice to specify bars sizes that are not too similar in size to reduce the potential mistake during construction.

In summary, the bar reinforcement areas corresponding to positive and negative moments are as such:

$$A_{s\ positive} = 16\ (0.79\text{ in.}^2) = 12.64\text{ in.}^2$$

$$A_{s\ negative} = 16\ (1.27\text{ in.}^2) = 20.32\text{ in.}^2$$

5.6.7.3.6 Step 6: Check for Serviceability

See Section 5.6.6.3.6, serviceability check is similar.

5.6.7.3.7 Step 7: Check for Fatigue

Check for fatigue is same as the example shown in Section 5.6.6.3.7, and therefore is not repeated here.

5.6.7.3.8 Step 8: Perform shear Design

Calculate Factored Shear

Unfactored shears and associated moments for the right face of the left column, Frame Element 34, extracted from CSiBridge output are summarized in Table 5.6.7-3.

Table 5.6.7-3 Maximum Shears and Associated Moments for Interior Face of Column -Frame Element 34

Shears (kip)	Associated Moments (kip-ft)
$V_{DC\_34} = 344.5$	$M_{DC\_34} = -1127.5$
$V_{DW\_34} = 47.3$	$M_{DW\_34} = -82.6$
$V_{HL\text{-}93\_34} = 300.6$	$M_{HL\text{-}93\_34} = -196.3$
$V_{Permit\_34} = 510.1$	$M_{Permit\_34} = -114.8$

Two load combinations, Strength I and II typically govern for shear design of bent caps. Factored shear demand and associated factored moments for the interior face of column are calculated as:

$V_{u\_34\_Strength\_I} = 1.25\ (V_{DC\_34}) + 1.5\ (V_{DW\_34}) + 1.75\ (V_{HL\text{-}93\_34}) = 1{,}027.6$ kip

$M_{u\_34\_Strength\_I} = 1.25\ (M_{DC\_34}) + 1.5\ (M_{DW\_34}) + 1.75\ (M_{HL\text{-}93\_34}) = -1{,}876.8$ kip-ft

$V_{u\_34\_Strength\_II} = 1.25\ (V_{DC\_34}) + 1.5\ (V_{DW\_34}) + 1.35\ (V_{Permit\_34}) = 1{,}190.2$ kip

$M_{u\_34\_Strength\_II} = 1.25\ (M_{DC\_34}) + 1.5\ (M_{DW\_34}) + 1.35\ (M_{Permit\_34}) = -1{,}688.3$ kip-ft

$V_{u\_34} = V_{u\_34\_Strength\_II} = 1{,}190.2$ kip ← Strength II governs

$M_{u\_34} = M_{u\_34\_Strength\_II} = -1{,}688.3$ kip-ft

Determine θ *and* β

Cross section of the drop cap is shown in Figure 5.6.7-13.

b_v is width of web taken as 90 in;

d_{cap} is depth of cap taken as 72 in.

d_e is effective depth taken as 69.5 in;

d_v is effective shear depth taken as the max (d_{v1}, 0.9 d_e, 0.72d_{cap});

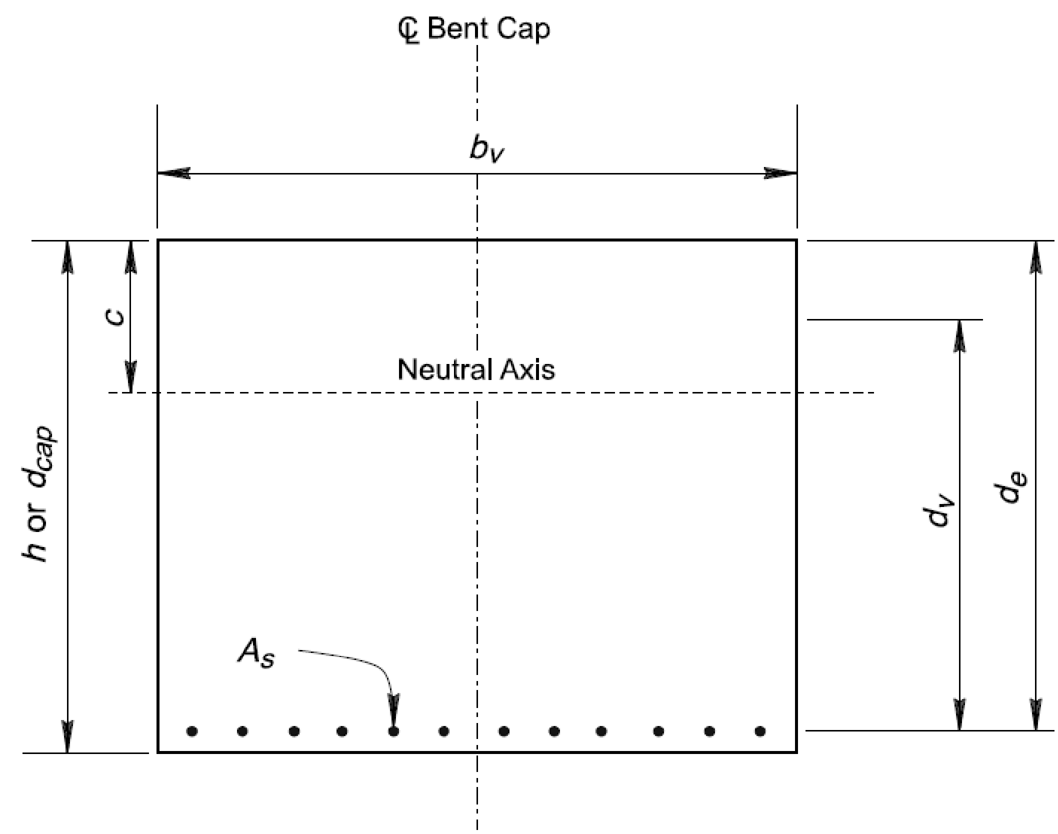


Figure 5.6.7-13 Cross Section of Drop Cap

Assuming $f_s = f_y$, we have

$$a = \frac{\left(A_{s\ negaitve} f_y - A_{s\ positive} f_y\right)}{0.85 f'_c b_v} = \frac{(20.32)(60)-(12.64)(60)}{(0.85)(4)(90)} = 1.51\text{ in.}$$

$$d_{v1} = d_e - \frac{a}{2} = 68.7\text{ in.}$$

d_v = max (d_{v1}, 0.9d_e, 0.72d_{cap}) = 68.7 in.

$$v_u = \frac{V_{u34}}{\phi_v b_v d_v} = \frac{1{,}190.2}{(0.9)(90)(68.7)} = 0.2\text{ ksi} \qquad \text{(AASHTO 5.7.2.8-1)}$$

Shear stress factor:

$$\frac{v_u}{f'_c} = \frac{0.2}{4} = 0.05$$

Assume (0.5cot θ = 1) and use absolute values for M_u and V_u for strain calculation, we have:

$$\varepsilon_x = \frac{\frac{|M_{u34}|}{d_v} + 0.5V_{u34}(\cot\theta)}{2E_s A_{s\ negative}} = \frac{\left(\frac{(1{,}688.3)(12)}{68.7} + 1{,}190.2\right)}{2(29{,}000)(20.32)} = 0.00126$$

(AASHTO B5.2-3)

From AASHTO Table B5.2-1, β = 2.23 and θ = 36.4° are obtained.

Use θ = 36.4° and recalculate ε_x as follows:

$$\varepsilon_x = \frac{\frac{|M_{u34}|}{d_v} + 0.5V_{u34}(\cot\theta)}{2E_s A_{s\ negative}}$$

$$= \frac{\left(\frac{(1{,}688.3)(12)}{68.7} + (0.5)(1{,}190.2)\left(\cot 36.4^o\right)\right)}{2(29{,}000)(20.3)} = 0.00094$$

From AASHTO Table B5.2-1, β = 2.23 and θ = 36.4° are obtained again. And convergence is reached.

Determine Shear Reinforcement

Concrete contribution to shear resistance:

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v \qquad \text{(AASHTO 5.7.3.3-3)}$$

$$V_c = (0.0316)(2.23)(1.0)(\sqrt{4})(90)(68.7) = 871.4 \text{ kips}$$

$$V_s = \frac{V_{u34}}{\phi_v} - V_c = \frac{1{,}190.2}{0.9} - 871.4 = 451.0 \text{ kips}$$

Required shear stirrups:

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v \cot\theta} = \frac{451.0}{(60)(68.7)(\cot 36.4^\circ)} = 0.081 \text{ in.}^2/\text{in.}$$

Check stirrup ratio with minimum allowed transverse reinforcement ratio per Article 5.7.2.5:

$$\left(\frac{A_v}{s}\right)_{\min} = 0.0316\lambda\sqrt{f_c'}\frac{b_v}{f_y} = 0.0316(1.0)\sqrt{4}\frac{90}{60} = 0.095 \text{ in.}^2/\text{in.}$$

Use stirrup ratio = 0.095 in.^{2}/in.

Compute stirrup spacing:

Try four legs of #6 bar reinforcing (A_b = 0.44 in.^{2}):

A_v = 4 (0.44 in.^{2}) = 1.8 in.^{2}

Required spacing, s = 1.8/0.095 = 18.95 in.

Check spacing requirement per AASHTO and CA 5.7.2.6:

$$\frac{v_u}{f_c'} = 0.05$$

$$s_{\max} = if\left(\frac{v_u}{f_c'} < 0.125,\ \min\ (18 \text{ in.},\ 0.8d_v),\ \min\ (12 \text{ in.},\ 0.4d_v)\right) = 18 \text{ in.}$$

For design, provide stirrup spacing of 18 in at the face of column.

Repeat the shear design steps as demonstrated above, for points along the drop cap length and produce a shear design chart. The design chart (Figure 5.6.7-14) shows the spacing requirements needed to satisfy the factored shear demand, V_u, as well as the minimum allowed transverse reinforcing ratios along the length of the bent cap.

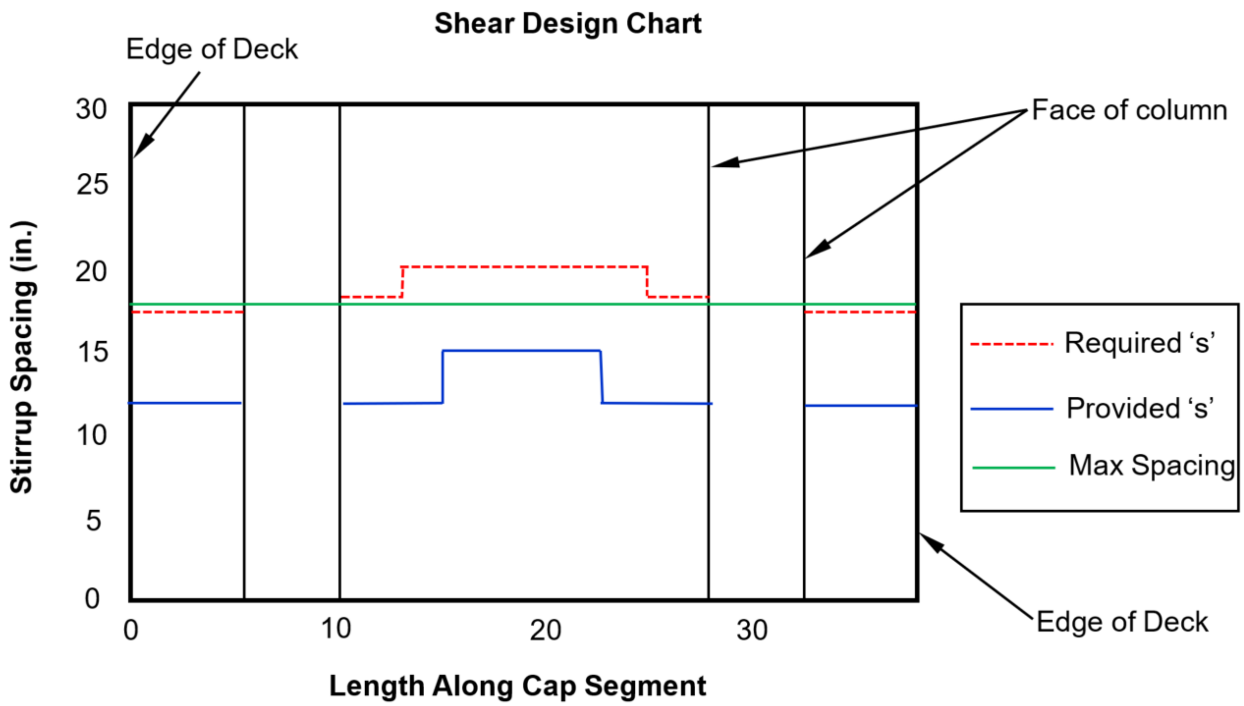


Figure 5.6.7-14 Shear Design Chart

5.6.7.3.9 Step 9: Determine Additional Details

Additional details are discussed as follows.

Spacing of Longitudinal Reinforcement

The clear distance between parallel bars shall not be less than larger of 1.5 (nominal diameter of bars), 1.5 (maximum size of coarse aggregate) and 1.5 inches (AASHTO 5.10.3.1.1). A limit on the maximum size coarse aggregate is appropriate; otherwise, it will control this clear distance. (2.2/1.5 = 1.46", the maximum aggregate).

For top negative flexural reinforcement, 16 #10 bars,

$$s_{longit\ min} = 1.5\ (1.27) = 1.9 \text{ in.}$$

The following calculation determines clear spacing between the negative flexural reinforcement, accounting for clear cover and approximate outside diameters of the #6 transverse reinforcement and #10 longitudinal bars:

$$s_{longit\ provided} = \frac{7.5(12) - 2 - 2 - 0.88 - 0.88 - 16(1.27)}{15} = 4.35 \text{ in.} \quad \text{OK}$$

$$> s_{longit\ \min} = 1.9 \text{ in.}$$

Side Face Reinforcement

Article 5.6.7 specifies that if d_l, distance from the extreme compression fiber to the centroid of extreme tension steel element of nonprestressed members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance d_l / 2 nearest the flexural tension reinforcing. The area of skin reinforcing, A_{sk} (in.^{2}/ft), of height on each side face shall satisfy the following equation. In this example, $d_e = d_l = 69.5$ in.

AASHTO required

$$A_{sk\ \min} = 0.012(d_e - 30) = 0.47 \text{ in}^2\text{/ft} \qquad \text{(AASHTO 5.6.7-3)}$$

The total area of longitudinal skin reinforcing (per face) need not exceed $A_{sk\ max}$.

$A_{ps} = 0$ in.^{2}

Our drop cap example does not contain prestressing.

$$A_{sk\ max} = \frac{\max\left[(A_{s\ positive}), (A_{s\ negative})\right] + A_{ps}}{4} = 7.62 \text{ in.}^2$$

The maximum spacing of the skin reinforcing shall not exceed $\frac{d_l}{6} = \frac{d_e}{6}$ or 12 in.

$$\frac{d_l}{6} = \frac{d_e}{6} = 11.6 \text{ in.}$$

Specify six #6 bars at each face at a spacing of 9 inches.

$$\text{Area provided } A_{sk} = \frac{0.44}{(12/9)} = 0.59 \text{ in.}^2\text{/ft} > 0.47 \text{ in.}^2\text{/ft} \qquad \text{OK}$$

SDC requires that the total longitudinal side face reinforcement in the bent cap shall be at least equal to $0.1A_{s\ positive}$ or $0.1A_{s\ negative}$ and shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches.

$$A_s^{sf} \geq \max\begin{Bmatrix} 0.1A_{cap}^{top} \\ 0.1A_{cap}^{bot} \end{Bmatrix} = 3.05 \text{ in.}^2 \qquad \text{(SDC 7.4.5.2-3)}$$

$$\text{Total area provided } A_s^{sf} = (12)(0.44) = 5.28 \text{ in.}^2 > 3.05 \text{ in.}^2 \qquad \text{OK}$$

Bent cap reinforcements are shown in Figures 5.6.5-15 to 5.6.5-17:

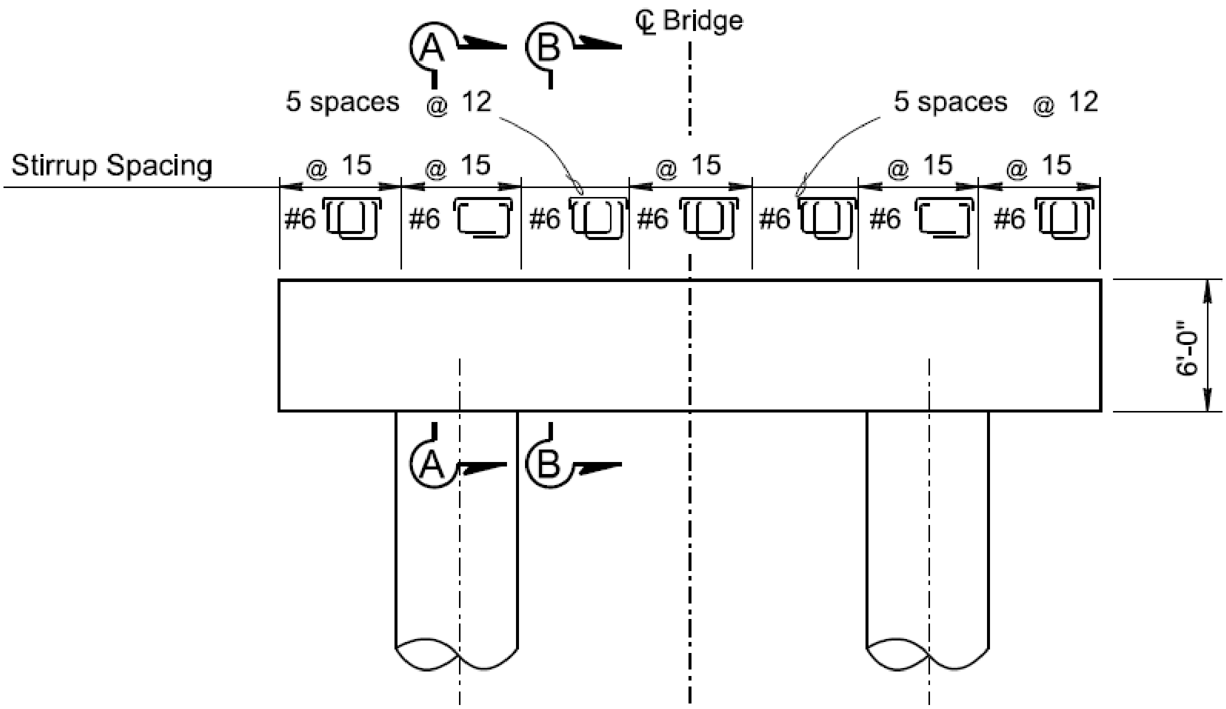


Figure 5.6.7-15 Elevation of Drop Cap Showing Stirrup Spacing

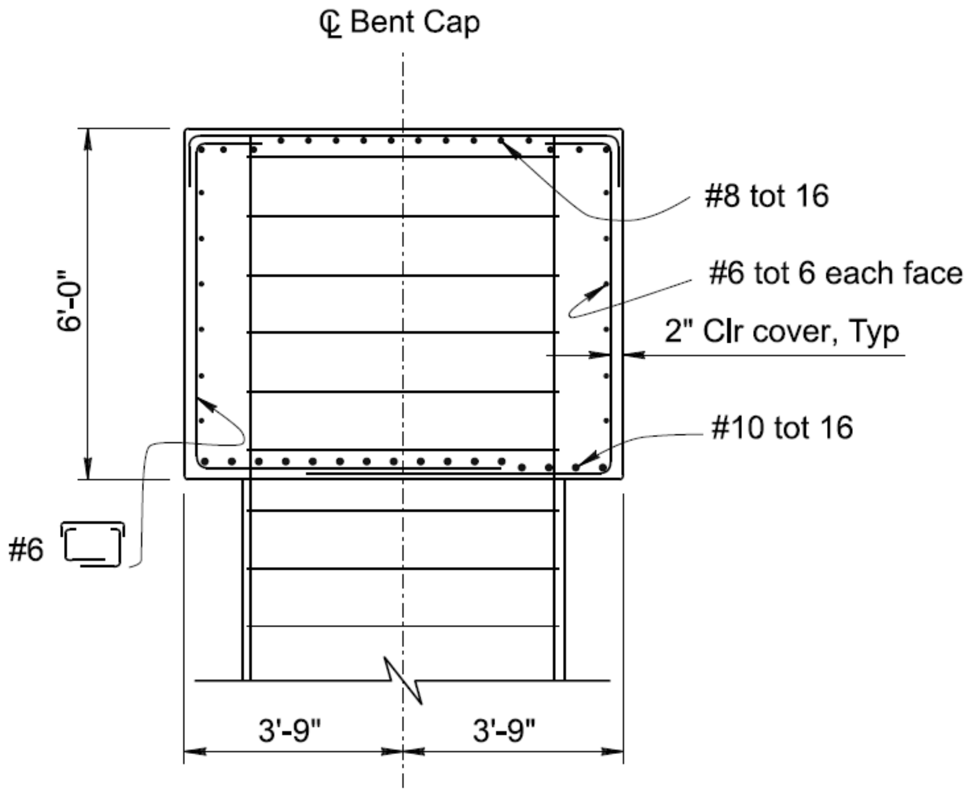


Figure 5.6.7-16 Section A-A

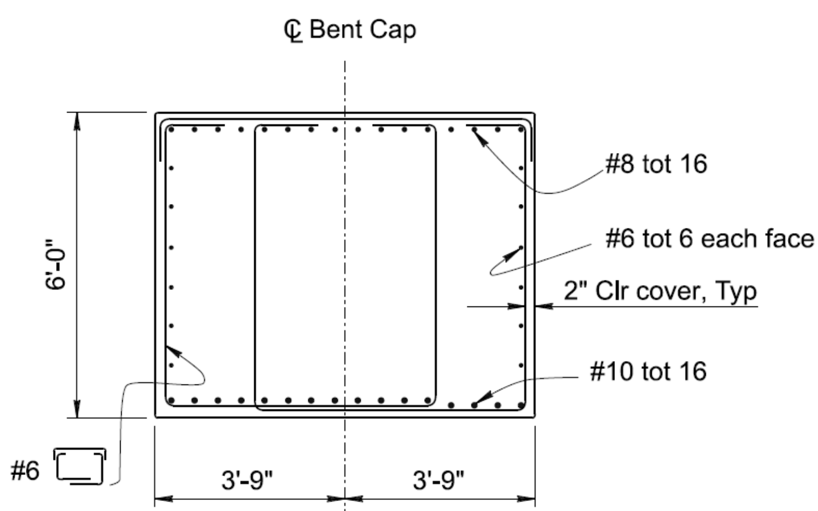


Figure 5.6.7-17 Section B-B

NOTATIONS

a = distance from the center of gravity (CG) of the superstructure to the bottom of column = height of the column + depth to CG of the bent cap

a = $c\beta_1$; depth of the equivalent stress block (in.)

A_{ps} = area of prestressing steel (in.^{2})

A_s = area of mild steel tension reinforcement (in.^{2})

A_s^{sf} = area of bent cap side face reinforcement (in.^{2})

A_{sk} = area of skin reinforcement per unit height on each side face (in^2)

A_s' = area of compression reinforcement (in.^{2})

A_v = area of shear reinforcement within a distance s (in.^{2})

b = top width of superstructure (in.)

b = width of the compression face of the member (in.)

b_f = flange width (in.)

b_{ledge} = ledge width (in.)

b_{stem} = stem width (in.)

b_v = effective web width taken as the minimum web width within the depth d_v (in.)

b_w = web width or diameter of a circular section (in.)

c = distance from the extreme compression fiber to the neutral axis (in.)

c = distance from the centerline (CL) of the columns to the edge of deck ($e + f$) (ft)

d = distance between the CL of columns (ft)

d = distance from the compression face to the centroid of tension reinforcement (in.)

d' = effective depth from extreme comp fiber to the centroid of compression reinforcement (in.)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest flexural reinforcement (in.)

d_{cap} = depth of the bent cap (in.)

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

d_{e1} = effective depth from extreme comp fiber to the centroid of the crack control reinforcement (in.).

d_{e2} = effective depth from extreme comp fiber to the centroid of tension reinforcement (in.)

d_{ledge} = ledge depth (in.)

d_p = distance from extreme compression fiber to the centroid of prestressing tendons (in.)

d_s = distance from extreme compression fiber to the centroid of non−prestressed tensile reinforcement (in.)

d_{stem} = stem depth (in.)

d_t = distance from the extreme compression fiber to the centroid of extreme tension steel (in.)

d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.)

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compression force due to flexure, it need not be taken to be less than the greater of $0.9d_e$ or $0.72h$ (in.)

e = distance from the CL of exterior girder to the edge of deck (EOD) along the skew (ft)

e_p = distance from the CL of exterior girder to the edge of deck (EOD) normal to the CL of the bridge (ft)

E_c = modulus of elasticity of concrete (ksi)

E_s = modulus of elasticity of steel reinforcement (ksi)

E_p = modulus of elasticity of prestressing steel (ksi)

f = distance from the CL of the column to the CL of the exterior girder (ft)

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) not including the effects of secondary moment, at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

f_{min} = minimum live load stress resulting from the Fatigue I load combination, combined with more serve stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; . positive if tension, negative if compression (ksi)

f_{ps} = average stress in prestressing steel at nominal bending resistance (ksi)

f_r = modulus of rupture of the concrete (ksi)

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi)

f_{ss} = tensile stress in steel reinforcement at the service limit state not to exceeds $0.6f_y$ (ksi)

f_y = specified minimum yield strength of reinforcement (ksi)

f'_c = compressive strength of concrete for use in design (ksi)

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi)

h_{cap} = bent cap depth (in.)

h = overall thickness or depth of the component (in.)

h_f = compression flange depth of an I or T member (in.)

I_g = moment of inertial of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4})

$L_{v,zero}$ = distance from the point of zero shear to the face of the support (in.)

M_{cr} = cracking moment (kip-in.)

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

M_{DC} = unfactored moment the section from dead load of structural components and nonstructural attachments (kip-in.)

M_{DW} = unfactored moment at the section from dead load of wearing surfaces and utilities (kip-in.)

M_n = nominal flexural resistance (kip-in.)

M_r = factored flexural resistance (kip-in.)

M_{ser} = moment at service limit state (kip-in.)

$M_{u(HL93)}$ = factored moment at the section from HL93 Vehicle (kip-in.)

$M_{u(\text{P}-15)}$ = factored moment at the section from the Permit Vehicle (kip-in.)

$|M_u|$ = absolute value of the factored moment, not to be taken less than $|V_u - V_p| d_v$ (kip-in.)

m = multiple presence factor

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

N = integer part of live load lanes

n = modular ratio

s = spacing of stirrups (in.)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

V_n = nominal shear resistance (kip)

V_p = component in the direction of the applied shear of the effective prestressing force (kip); positive if resisting the applied shear; typically zero for conventionally reinforced bent caps

V_r = factored shear resistance (kip)

V_s = shear resistance shear resistance provided by transverse reinforcement (kip) (kip)

V_u = factored shear (kip)

v_u = shear stress (ksi)

Y_t = distance from the neutral axis to the extreme tension fiber (in.)

Y_b = distance from the neutral axis to the extreme bottom fiber (in.)

α = angle of inclination of transverse reinforcement to longitudinal axis; typically 90° (degree)

θ = angle of inclination of diagonal compressive stresses (degree)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear

β_1 = stress block factor

β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

ε_x = longitudinal strain at the mid-depth of member

ε_t = net tensile strain

γ = load factor for Fatigue I

γ_1 = flexural cracking variability factor

γ_2 = prestress variability factor

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement

γ_e = exposure factor

ϕ = resistance factor

ϕ_f = resistance factor for moment

ϕ_c = resistance factor for axial load

ϕ_v = resistance factor for shear

Δf = live load stress range (ksi)

REFERENCES

1. AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications,* 8th Edition, American Association of State Highway and Transportation Officials, Washington, DC.

2. Caltrans. (2019a). *California Amendments to AASHTO LRFD Bridge Design Specifications—8th Edition*, California Department of Transportation, Sacramento, CA.

3. Caltrans. (2019b). Caltrans *Seismic Design Criteria, Version 2.0*, California Department of Transportation, Sacramento, CA.

4. CSi. (2021). *CSiBridge 23, Bridge analysis software*, Computer & Structures, Inc. Walnut Creek, CA.

5. Caltrans. (2022). *CTBridge,* (*v 1.9.6*), *Caltrans Bridge Analysis and Design Software,* California Department of Transportation, Sacramento, CA.

6. Viathor. (2016). *VBent, (v3.3.2), Bridge Engineering software*, Viathor, Inc. Orangevale, CA.